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CORTE MADERA CREEK SEDIMENTATION STUDY

Numerical Model Investigation

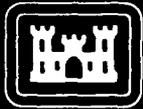
by

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Hydraulics Laboratory

DEPARTMENT OF THE ARMY

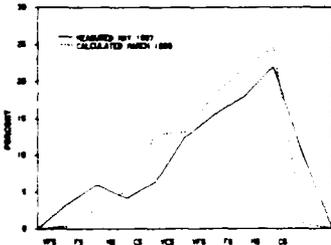
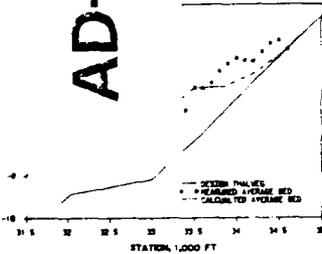
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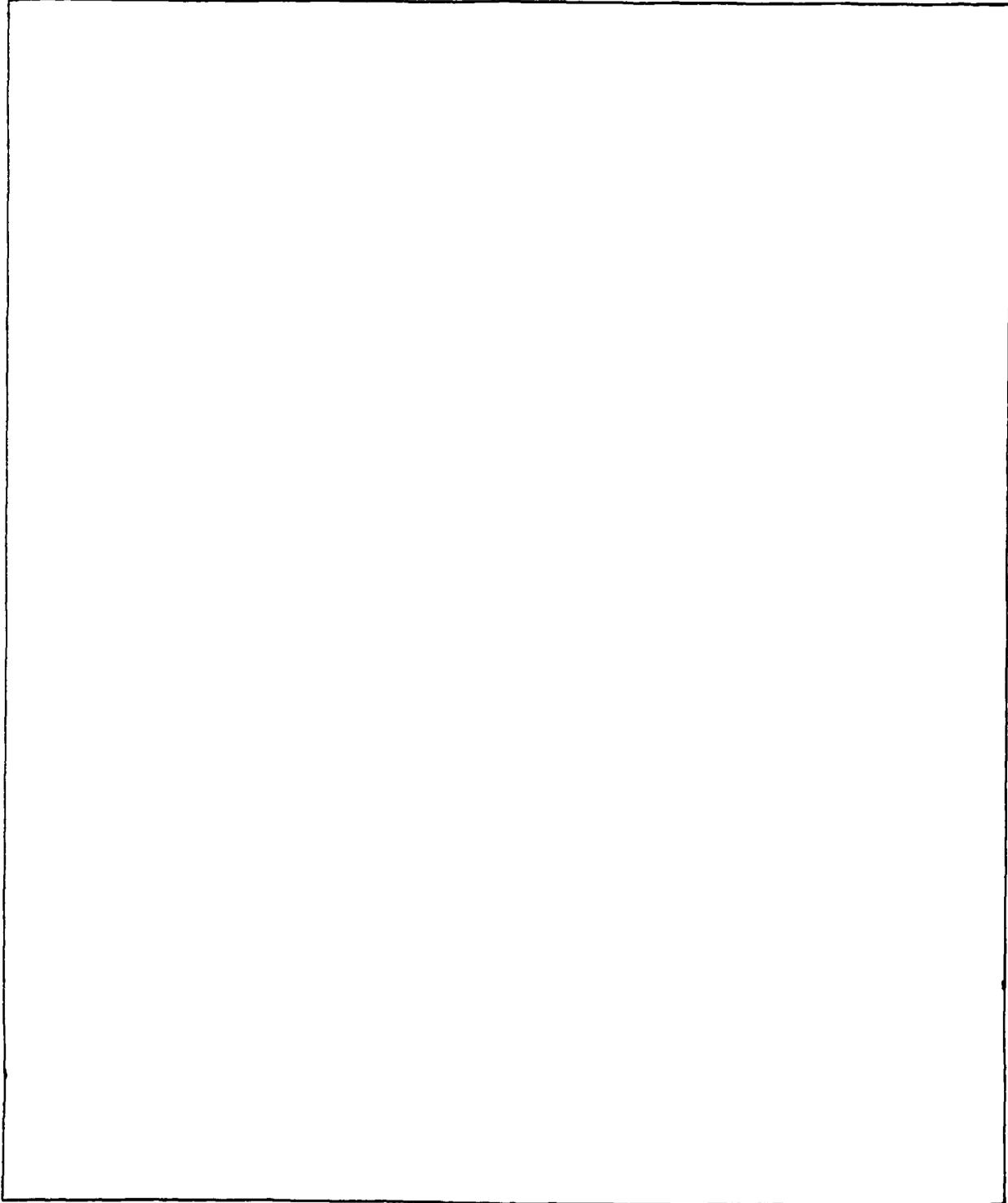
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PREFACE

The numerical model investigation of Corte Madera Creek, reported herein, was conducted at the US Army Engineer Waterways Experiment Station (WES), at the request of the US Army Engineer District, Sacramento (SPK).

This investigation was conducted during the period January 1988 to November 1988 in the Hydraulics Laboratory of WES, under the direction of Mr. Frank A. Herrmann, Jr., Chief of the Hydraulics Laboratory, Mr. Marden B. Boyd, Chief of the Waterways Division, and Mr. Michael J. Trawle, Leader of the Math Modeling Group. Mr. William A. Thomas provided general guidance and review. The Project Engineer for this study was Mr. Ronald R. Copeland, Math Modeling Group, who also prepared this report. Technical assistance was provided by Mrs. Peggy Hoffman, Math Modeling Group. This report was edited by Mrs. Marsha Gay, WES Information Technology Laboratory.

Mr. Charles S. Mifkovic served as the Hydraulic Project Engineer in SPK, providing valuable contributions and review during the course of the study.

COL Dwayne G. Lee, EN, is the Commander and Director of WES. Dr. Robert W. Whalin is Technical Director.



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CONTENTS

	<u>Page</u>
PREFACE.....	1
CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT.....	3
PART I: INTRODUCTION.....	5
The Prototype.....	5
Purpose of the Numerical Model Study.....	6
PART II: THE MODEL.....	8
Description.....	8
Channel Geometry.....	9
Hydrographs.....	10
Downstream Water-Surface Elevation.....	12
Bed Material.....	12
Channel Roughness.....	14
Sediment Inflow.....	18
Transport Function.....	19
PART III: MODEL ADJUSTMENT AND CIRCUMSTANTIATION.....	22
Adjustment to Deposition Surveys.....	22
Model Circumstantiation.....	24
Sensitivity Study.....	31
PART IV: STUDY RESULTS.....	32
Standard Project Flood.....	32
Average Annual Deposition.....	33
Initial Deposition in Earthen Channel.....	34
Design Conditions: Base Test.....	35
Sensitivity Studies.....	36
Effect of Sediment Trap.....	40
Design Roughness Coefficients.....	41
PART V: CONCLUSIONS AND RECOMMENDATIONS.....	43
Conclusions.....	43
Recommendations.....	45
REFERENCES.....	46
TABLE 1	
PLATES 1-20	
APPENDIX A: DESCRIPTION OF TABS-1 COMPUTER PROGRAM.....	A1
APPENDIX B: DETERMINATION OF DESIGN ROUGHNESS COEFFICIENTS.....	B1
TABLE B1	

CONVERSION FACTORS, NON-SI TO SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

<u>Multiply</u>	<u>By</u>	<u>To Obtain</u>
cubic feet	0.02831685	cubic metres
cubic yards	0.7645549	cubic metres
feet	0.3048	metres
inches	2.54	centimetres
miles (US statute)	1.609347	kilometres
square miles	2.589998	square kilometres
tons (2,000 pounds, mass)	907.1847	kilograms

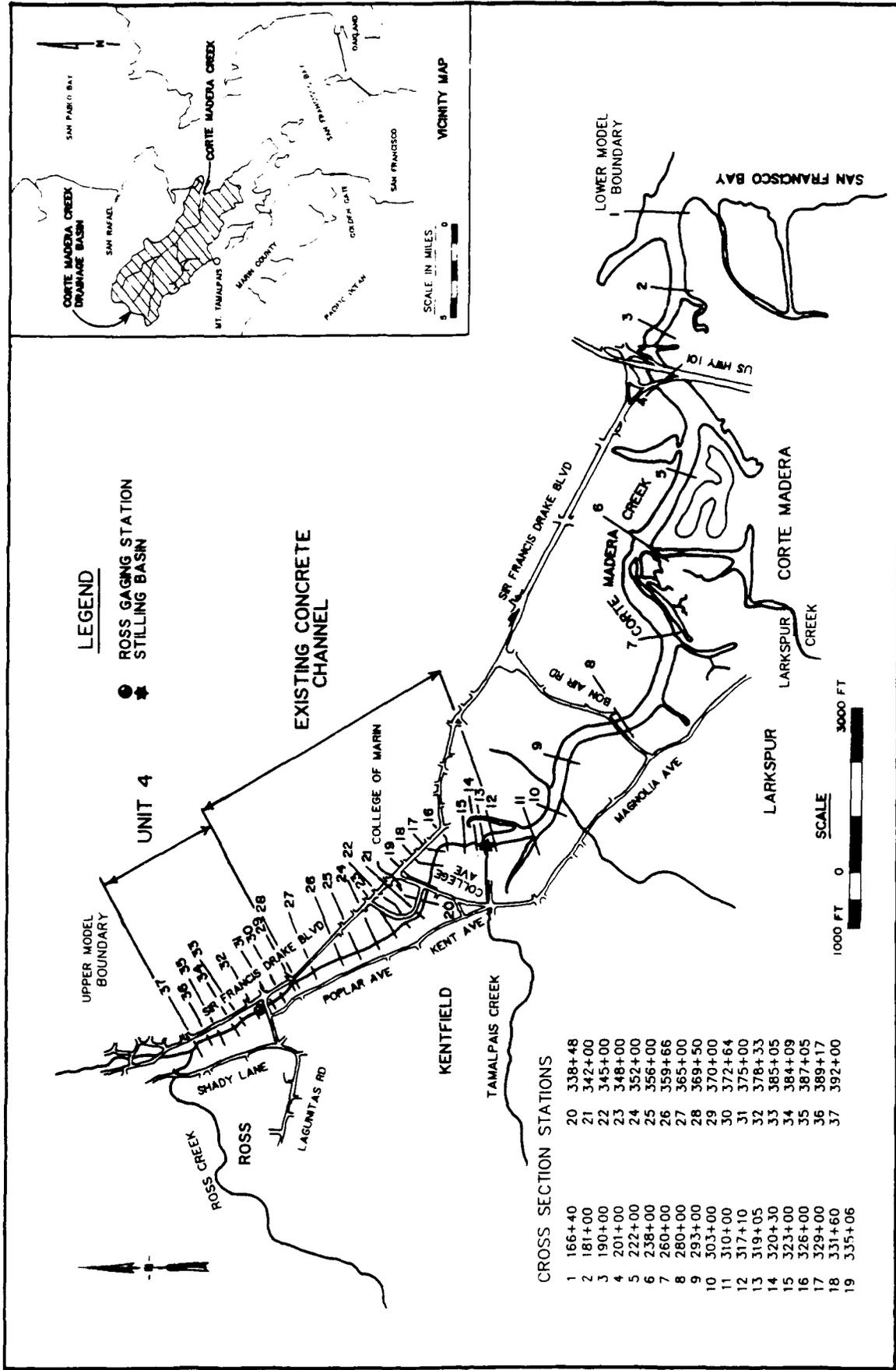


Figure 1. Cross section locations

CORTE MADERA CREEK SEDIMENTATION STUDY

Numerical Model Investigation

PART I: INTRODUCTION

The Prototype

1. Corte Madera Creek drains an area of approximately 28 square miles* in Marin County, California. The creek discharges into San Francisco Bay about 9 miles north of the Golden Gate. In 1972 the US Army Corps of Engineers completed three units of a proposed four-unit project on Corte Madera Creek extending from San Francisco Bay through the cities of Corte Madera, Larkspur, Kentfield, and Ross, a distance of about 4.5 miles (Figure 1). The first 2.2 miles of the project consist of an earth channel, dredged to el -12.0** with a bottom width of 80 ft and side slopes of 1V:6H. The next 0.7 mile of earth channel has a bottom slope of 0.0007, a 30-ft bottom width, and 1V:6H side slopes. The next mile of the project consists of a 33-ft-wide concrete channel with a stilling basin at the downstream end. The first 1,000 ft of the channel has a mild slope of 0.0007. The remainder of the concrete channel has a steep slope of 0.0038, and is designed for supercritical flow. These segments have been constructed. The final unit, Unit 4, of the project was to be an additional 3,000 ft of concrete channel. Construction of Unit 4 was delayed due to litigation, environmental concerns, and strong public opposition. An alternative plan for Unit 4 was developed that would have preserved the ecological character of the creek, and after extensive public coordination the plan received public support. However, in 1980, before the plan was approved, the Marin County Board of Supervisors withdrew local sponsorship, and engineering and design work was suspended by the Corps of Engineers.

2. Existing flooding from Corte Madera Creek is due primarily to the insufficient channel capacity in the Unit 4 reach. Flows greater than

* A table of factors for converting Non-SI units of measurement to SI (metric) units is presented on page 3.

** All elevations (el) cited herein are in feet referred to National Geodetic Vertical Datum (NGVD).

approximately 3,000 cfs overtop the right descending bank from upstream of the Lagunitas Bridge to the upstream end of the existing concrete channel. These flood flows proceed down Poplar and Kent Avenues inundating areas adjacent to the existing channel improvements in Ross, Kentfield, and the College of Marin, Kentfield. Channel capacity in the downstream reaches of the concrete channel is reduced due to the accumulation of sands and gravels. This accumulation reduces conveyance and increases the composite channel roughness.

3. The largest recorded flood flow on Corte Madera Creek occurred in January 1982. This flood had an estimated peak of 7,200 cfs at the Ross gage and a recurrence frequency greater than 100 years. Another flood occurred in March 1983, with an estimated peak of 3,480 cfs and a recurrence interval of about 6 years. This event was the third largest flood of record. Both of these floods resulted in damages to homes and businesses adjacent to Corte Madera Creek. In December 1983, the Marin County Board of Supervisors requested the Corps of Engineers to reinitiate the project.

4. After extensive local coordination, engineering analysis of the data, and experience obtained from the recent floods, a new plan for Unit 4 was developed by the US Army Engineer District (USAED), Sacramento, consisting of channel improvements, floodwalls, and a sediment trap (Plate 1). A 270-ft extension of the concrete channel is proposed with a sloping concrete drop structure at its upstream end. An earthen trapezoidal channel will extend about 350 ft upstream to the Lagunitas Road Bridge where it will join a 300-ft-long sediment trap. The sediment trap will be dredged 4 ft into the creek bed. The Lagunitas Road Bridge and approaches will be raised. Upstream from Lagunitas Road Bridge, for about 1,200 ft, floodwalls will be constructed on both the right and left overbanks. Channel improvements through the remainder of the project reach would consist of gabions and crib walls, placement of riprap, and planting of vegetation in selected areas to protect the channel banks from erosion and prevent undercutting of the floodwalls and channel banks. Channel wall heights in the existing concrete channel will be increased where necessary to meet freeboard requirements.

Purpose of the Numerical Model Study

5. The numerical model study was performed to evaluate the deposition pattern in the concrete channel and to determine the effectiveness of the

sediment trap in reducing or eliminating the deposition problem. The effect of the accumulated sediment on channel roughness was evaluated. Critical to this study was the adequacy of channel wall heights in the existing concrete channel. The numerical model used in this study, TABS-1, is primarily a sediment transport model and does not have the refinements of a backwater model such as HEC-2 for calculation of design water-surface elevation. However, the numerical model may be used to determine the extent of the accumulated sediment under various conditions and the effect of the deposit on channel roughness. The model was also used to evaluate the stability of the proposed earthen channel between the concrete channel and the sediment trap, and to determine average annual deposition quantities.

PART II: THE MODEL

Description

6. The TABS-1 one-dimensional sedimentation program was used to develop the numerical model for this study. Development of this computer program was initiated by Mr. William Thomas at the US Army Engineer District, Little Rock, in 1967. Further development at the US Army Engineer Hydrologic Engineering Center (USAEHEC) by Mr. Thomas produced the widely used HEC-6 generalized computer program for calculating scour and deposition in rivers and reservoirs (USAEHEC 1977). Additional modification and enhancement to the basic program by Mr. Thomas at the US Army Engineer Waterways Experiment Station (WES) led to the TABS-1 program currently in use (Thomas 1980, 1982). The program produces a one-dimensional model that simulates the response of the riverbed profile to sediment inflow, bed material gradation, and hydraulic parameters. The model simulates a series of steady-state discharge events and their effects on the sediment transport capacity at cross sections and the resulting degradation or aggradation. The program calculates hydraulic parameters using a standard-step backwater method assuming subcritical flow. The program assigns critical depth for water-surface elevation if the backwater calculations indicate transitions to supercritical flow. However, for supercritical flow, hydraulic parameters for sediment transport are calculated assuming normal depth in the channel. A more detailed description of the program capabilities is found in Appendix A.

7. For numerical sedimentation models to completely simulate the behavior of a stream channel, computations would have to account for all of the basic processes of sedimentation: erosion, entrainment, transportation, deposition, and compaction of both the bed and the streambanks for the complete range of particle sizes found in nature. The state of the art has not yet advanced to such a complete simulation. The computer program used in this study, TABS-1, is a state-of-the-art program for use in mobile bed channels. It is designed to calculate aggradation and degradation of the streambed profile. When applied by experts using good engineering judgement, the TABS-1 program will provide good insight into the behavior of mobile bed channels such as Corte Madera Creek.

8. Particle sizes from sand to gravel are involved in Corte Madera

Creek, which complicates the simulation because particle size controls the fundamental processes in river sediment behavior. The time scale of interest is from a single flood event to the life of the project. The long-term trends can be evaluated from a statistical analysis of the gage records, but a great deal of variation in water and sediment runoff occurs from one storm event to the next because of the stochastic nature of the hydrologic cycle. The approach for bridging these gaps is to formulate (a) a procedure that includes the statistical nature of the boundary conditions - the uncertainty in forecasting future hydrology and sediment yield is probably more significant than gaps in modeling the physics of the mobile boundary processes so far as the accuracy of results is concerned; and (b) a computer program that emulates the physical processes in the project reach sufficiently well to quantify how the sedimentation processes will respond to changes in the boundary conditions and/or to changes in the project geometry or roughness.

9. Although the sedimentation processes are complex, procedures for describing most of them have been published. The TABS-1 computer program includes those procedures. Where gaps exist between the available procedures, TABS-1 contains logic that bridges those gaps. In summary, it is state-of-the-art technology for calculating the aggradation and degradation in mobile bed channels, and because it has given reliable results at similar projects, it is expected to give reliable answers to the questions being addressed here.

Channel Geometry

10. The numerical model extends from sta 166+00 near the mouth of Corte Madera Creek at San Francisco Bay to sta 392+00, which is just downstream from the confluence of Ross Creek. The channel geometry for the historical simulation was based on cross sections from HEC-2 backwater models provided by the Sacramento District. These sections considered the channel to be dredged to design dimensions. Channel geometry for the design channel (Unit 4) was also based on HEC-2 cross sections provided by the Sacramento District. These cross sections incorporate channel design changes as described in the Design Memorandum (USAED, Sacramento, 1987). Reach lengths between cross sections are generally greater in a TABS-1 model than in a HEC-2 model. Reach lengths in this model were generally about 2,000 ft downstream from the stilling basin and about 300 ft upstream. Cross-section locations are shown in Figure 1.

Hydrographs

11. Discharge hydrographs are simulated in the numerical model by a series of steady-state events. The duration of each event is chosen such that changes in bed elevation due to deposition or scour do not significantly change the hydraulic parameters during that event. At relatively high discharges, durations need to be short; time intervals as low as 1 hour were used for Corte Madera Creek. At low discharges, the time interval may be extended. Time intervals up to 3 days were used in this study.

12. A hydrograph simulated by a series of steady-state events of varying durations is called a histogram. The histogram used in the adjustment and circumstantiation of the study was based on historical data from the US Geological Survey's (USGS) gage on Corte Madera Creek at Ross. The gage is located at sta 379+50, which is about 250 ft upstream from Lagunitas Road Bridge. Mean daily discharges greater than 100 cfs between October 1972 and September 1986 were used to develop a historical histogram. Sediment transport is negligible at discharges below 100 cfs. In addition to mean daily flows, USGS reported 57 peak discharges during this 14-year period. Histogram events were adjusted to account for the increased sediment transport potential during high flow events. Reported peaks were assigned a duration of 4 hours and the corresponding mean daily flow was reduced to maintain the same runoff volume. The 4-hour duration was chosen based on durations of peak flows from December 1955 and March 1983 flood hydrographs. Actual historical flood hydrographs were used for the major floods of 3-5 January 1982, 12-13 March 1983, and 12-21 February 1986. The historical histogram at the Ross gage is shown in Plate 2. Days with mean daily flows less than 100 cfs are excluded from the histogram so that the abscissa is time discontinuous. The beginning of each water year and the occurrence of the three major flood peaks during the period are marked on the histogram in Plate 2.

13. Flow breaks out of the Corte Madera channel just upstream from Lagunitas Road Bridge and just upstream of the existing concrete channel when flow exceeds about 3,000 cfs. A breakout discharge rating curve for each of these locations was developed by the Sacramento District based on high-water marks from the January 1982 flood (Plate 3). These rating curves were used in the numerical model to remove flow from the channel downstream from the

breakouts. Breakout flows return to the channel downstream of the stilling basin near the confluence with Tamalpais Creek.

14. Downstream tributary inflow hydrographs were included in the model for the January 1982 flood. Tributary inflow was determined by the Sacramento District from storm reconstitution studies. A conclusion of that study was that tributary contributions to the flow at the Ross gage would be insignificant at discharges less than 4,000 cfs. Tributary inflow points were at College Avenue, Tamalpais Creek, an unnamed tributary near sta 303+00, and Larkspur Creek.

15. The design flood hydrograph has a 100-year frequency peak (6,900 cfs at the Ross gage) and has the same shape as the January 1982 flood hydrograph. The histogram used in the numerical model was obtained by reducing each steady-state discharge in the January 1982 flood histogram by the ratio of the flood peaks (0.958). The design flood included tributary inflow when the discharge at the Ross gage exceeded 4,000 cfs. The longitudinal distribution of water discharge at the design flood peak is shown in the following tabulation:

Location	Sta	Discharge cfs
Upstream boundary	392+00	6,900
Downstream of College Avenue	335+06	6,950
Downstream of Tamalpais Creek	317+10	7,170
Downstream of unnamed creek	303+00	7,610
Downstream of Larkspur Creek	244+00	8,800

Note: A December 1988 hydrologic analysis of the 100-year flood by the Sacramento District resulted in slight variations in discharge and locations from those listed; however, the differences are small and would not affect the results of this report.

16. The effects of flows antecedent and subsequent to the design flood were considered in the numerical model tests. The design histogram included discharge events from the 1982 flood season, with the January 1982 flood replaced by the design flood. The design histogram is shown in Plate 4.

17. The Standard Project Flood (SPF) histogram was based on hydrographs calculated for Corte Madera Creek by the San Francisco District (USAED, San Francisco, 1966). The peak flow for this flood is 7,500 cfs at the Ross

gage, and 9,700 cfs at San Francisco Bay. In the numerical model, differences in the SPF hydrograph at the Ross gage and San Francisco Bay were attributed to tributary inflow from Tamalpais Creek. Time intervals of 1 hour were used for most of the SPF histogram. The histogram is shown in Plate 5.

18. The flow duration histogram, developed to determine average annual deposition, was based on discharge duration data from the Ross gage. The period of record was 32 years (1952-1984). USGS data included durations for discharges as high as 2,500 cfs. Discharge durations for larger events were calculated using hydrographs from December 1955, January 1982, and March 1983. An annual histogram was developed by multiplying the fraction of time for which a certain duration was exceeded by 365 days to obtain representative durations for discharge. A symmetrical shape was assumed for the histogram. The annual flow duration histogram is shown in Plate 6. Flows above 100 cfs were exceeded about 7 percent of the time; thus, the annual flow duration histogram used in this study had a duration of about 24 days.

Downstream Water-Surface Elevation

19. Starting water-surface elevations at the downstream end of the numerical model were determined using data from the National Oceanic and Atmospheric Administration (NOAA). The mean tide level at San Quentin (el 0.5) was used as the downstream water-surface elevation for the design, SPF, and annual flow duration histograms, and most of the historical histogram. Actual measured tide levels at San Francisco, adjusted to the San Quentin gage, were used for the downstream water-surface elevations during the January 1982 and March 1983 flood events. An additional test was conducted using the design histogram with a downstream water-surface elevation of 3.24, which is the mean higher high water elevation for January 1982.

Bed Material

20. The numerical model requires that an initial volume and gradation be specified for the bed material. This was accomplished in the concrete channel reach by specifying a bed material depth of zero for the initial condition. The gradation of deposited material is then calculated by the model's sorting and settling algorithm. The initial bed gradation for the reach

downstream from the concrete channel was based on a sample collected 2,000 ft downstream from the stilling basin (sta 300+00) in April 1984. The D_{50} of this material (the particle size of which 50 percent is finer) was 0.2 mm. A single sample is generally insufficient to determine the gradation of a reach. However, in this case, it was deemed adequate because (a) this reach is essentially a depositional reach and the gradation of the active layer will be determined from the inflowing sediment load and not the bed reservoir; and (b) this reach was included in the numerical model primarily for the purpose of determining water-surface elevation at the downstream end of the concrete channel, and therefore, accurate simulation of the bed profile was of secondary importance. Simulating bed profile changes in the downstream reach would require the inclusion of silts and clays and the effects of sediment deposition and resuspension due to tidal action in the model, which is beyond the scope of this investigation. The bed material gradation in the natural channel, upstream from the concrete channel, was determined from the average of four samples collected in January 1985 by the Sacramento District (Figure 2). These samples were taken between sta 372+00 (about 300 ft downstream from Lagunitas Road Bridge) and sta 382+00 (about 250 ft upstream from the Ross gage). This gradation was used to determine equilibrium sediment transport capacity at the upstream boundary of the numerical model as described in the

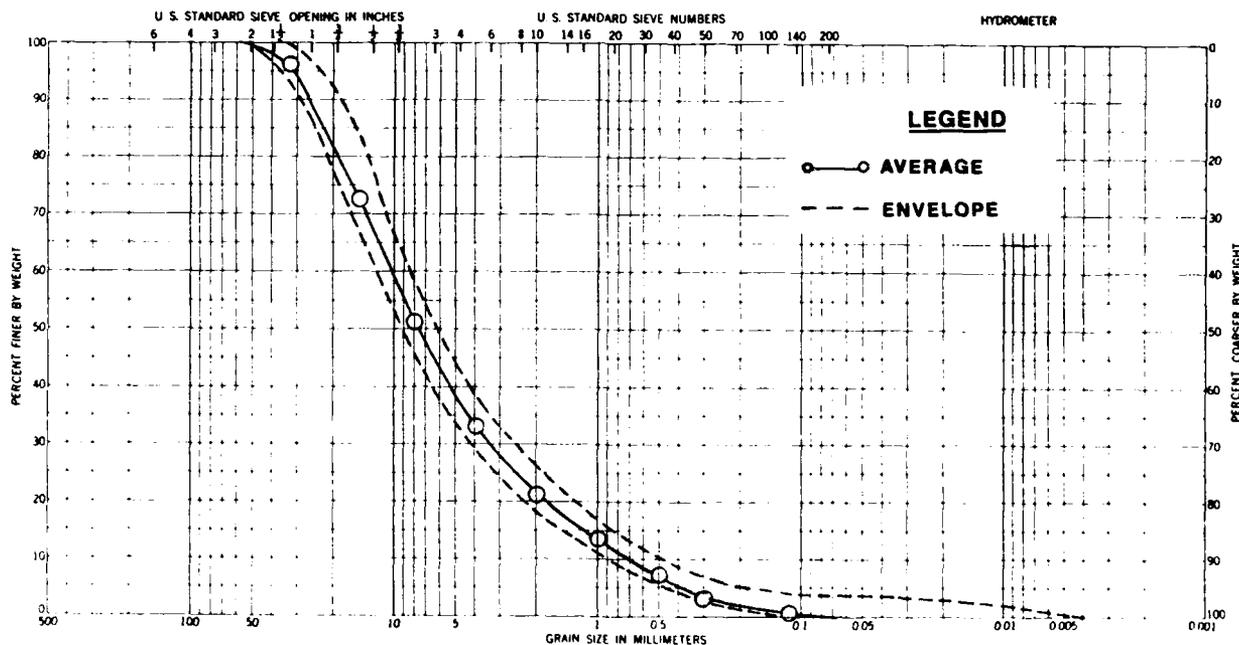


Figure 2. Bed material gradations, vicinity of Ross gage

section on sediment inflow. Initially this gradation was also used in the model for the channel upstream from the concrete-lined section. However, during the adjustment phase of the study, significant degradation was calculated by the model for large flood events, a condition that does not conform to field observations. Several adjustments to the initial gradation were tested, but in each test, the bed degraded during high flow and did not aggrade with subsequent low flows. This behavior in the model is attributed to the model's inability to adequately simulate the effect of cobbles and boulders on the armor layer at very high flows. Material larger than 64 mm is not considered in the armor calculations in this version of TABS-1. Since it was not considered reasonable to allow the streambed to act as a source of material for the deposition in the concrete channel, the bed material reservoir in this reach of channel was assigned a depth of zero. With this initial condition, the model calculated a bed material depth and gradation based on the hydraulic parameters and the incoming sediment load.

Channel Roughness

21. Hydraulic roughness is influenced by grain size or bottom roughness, bank or sidewall roughness, bed form, water depth, changes in channel shape, and changes in flow direction or distribution due to bends and confluences. In the one-dimensional numerical model these effects are accounted for by the Manning's roughness coefficient. Acceleration and deceleration of flow are accounted for with expansion and contraction coefficients. The roughness coefficient may vary significantly with discharge and time. The influence of grain or bottom roughness on hydraulic losses is known to decrease with increases in depth. Resistance due to bed forms can decline dramatically when the bed forms are washed out and replaced by a plane bed. An attempt to account for these problems was made in this study by developing an algorithm that calculated composite roughness coefficients based on roughness attributed to the bed, sidewalls, and bed movement.

22. Determining composite roughness in the concrete channel is complicated by the accumulation of sand and gravel on the channel bottom. A composite roughness coefficient was calculated using the following formula:

$$n = \left(\frac{P_w n_w^{1.5} + P_b n_b^{1.5}}{P_w + P_b} \right)^{2/3} \quad (1)$$

where

P_w = perimeter of the wall
 n_w = roughness coefficient of the wall
 P_b = perimeter of the bed
 n_b = roughness of the bed

23. The bed grain roughness was calculated using the Limerinos equation (Limerinos 1970):

$$n_b = \frac{0.0926(R'_b)^{1/6}}{1.16 + 2.03 \log \left(\frac{R'_b}{D_{84}} \right)} \quad (2)$$

where

R'_b = hydraulic radius of the bed attributed to grain roughness
 D_{84} = particle size of which 84 percent of the bed is finer

In order for sand and gravel to influence the bottom roughness, a minimum bed thickness was required in the model. If this requirement was not met, the model assigned a D_{84} value that is representative of concrete. The minimum thickness was the larger of 4 mm or two times the grain size for which the percent coarser fraction covered the bed to a thickness of two grain diameters.

24. The Limerinos equation accounts primarily for the grain roughness. Additional bed roughness can be caused by the form roughness that occurs with a movable bed. Calculations using the Brownlie equation (Brownlie 1983) for upper regime flow showed an increase in the Manning's roughness coefficient of 0.010 due to form roughness. In the numerical model, the bed roughness coefficient was increased to account for form roughness if both the minimum bed thickness and the critical shear stress were exceeded. The Shields equation was used to determine critical shear stress.

$$\tau_c = \theta(\gamma_s - \gamma_w)D_{50} \quad (3)$$

where

- τ_c = critical shear stress
- τ = Shields parameter
- γ_s = specific weight of sediment
- γ_w = specific weight of water

Various investigations have established a range for the Shields parameter between 0.03 and 0.06 when median grain diameter is used in the equation. In order to provide a continuous transition for the increase in roughness coefficient for form roughness, the following procedure was adopted. If the calculated shear stress was less than critical shear stress using a Shields parameter of 0.03, then the bed was assumed to be immobile and no adjustment was made to the Limerinos bed roughness. If the calculated shear stress was greater than the critical shear stress using a Shields parameter of 0.06, then the Limerinos bed roughness was increased by 0.010 to account for form roughness due to the movable bed. The roughness increase was linearly interpolated for conditions between these limits.

25. The sidewall roughness in the composite roughness equation accounted for the roughness height of the wall itself. An appropriate roughness coefficient was determined by simulating high-water marks from the January 1982 flood. The wall roughness should be somewhat higher than for finished concrete because of tubeworm and barnacle deposits. A value of 0.018 was chosen.

26. Initially, calculated water-surface elevations in the lower portion of the concrete channel were significantly lower than the reported high-water marks for the January 1982 flood. To correct this discrepancy, the roughness was increased by 0.004 to account for losses due to channel bends in the downstream reach, below sta 342+00. With this revision, calculated water-surface elevations and high-water marks were much closer, but the calculated values were still slightly lower than high-water marks. These water-surface comparisons are shown in Figure 3. Differences in reported and calculated values for the high-water marks taken along the channel may be attributed primarily to losses at bridges at College Avenue and College of Marin or wave action. In addition, some of the high-water marks were taken a sufficient distance from

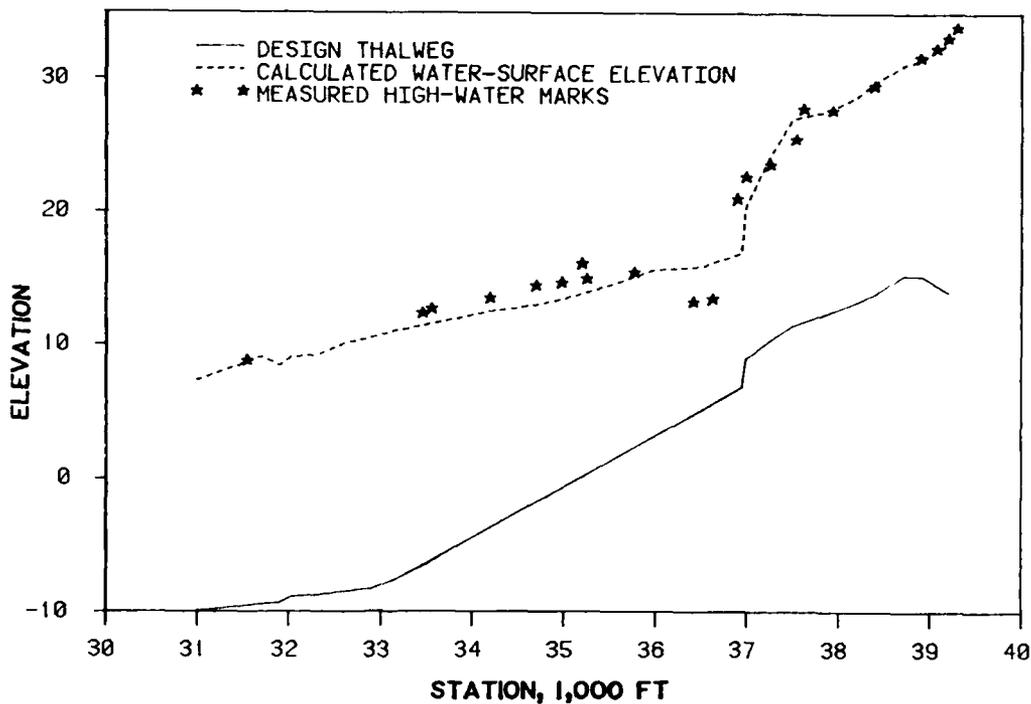


Figure 3. Maximum water-surface elevations, 4 January 1982

the channel that they may have represented overbank conditions more than the channel flow conditions. The assigned roughness coefficients were within the upper range used in engineering practice for this type channel, and therefore, further increases in roughness coefficients were deemed unreasonable.

27. The roughness algorithm developed for the concrete channel was not applicable for the reach downstream from the stilling basin. The model did not simulate the silt and clay fractions that are an important part of the bed downstream and must be considered in the calculation of D_{84} . A constant roughness coefficient of 0.045 was assigned to this reach for this study. This value was determined by matching the most downstream high-water mark from the 1982 flood. This relatively high value was required in the sediment model to account for reduced conveyance in the prototype due to deposition of silts and clays (not simulated in the numerical model) in the reach downstream from the stilling basin. A lower roughness coefficient should be used in a back-water model that accounts for complete deposition.

28. The roughness algorithm was applied in the Unit 4 reach of the natural channel upstream from the concrete channel. A shortcoming of this procedure developed because it became necessary to assign a bed thickness of zero for initial conditions in the numerical model. Until a sufficient bed thickness developed during the simulation, the numerical model considered the

bed roughness to be the same as concrete. This problem was minimized by assigning higher than normal values to the bank roughness coefficients. Bank roughness coefficients in the Unit 4 reach were determined during the water-surface adjustment study. Composite roughness coefficients ranged between 0.050 at high flows and 0.037 at low flows.

Sediment Inflow

29. Measured sediment inflow data for Corte Madera Creek are inadequate to determine a reliable sediment inflow rating curve for the entire range of discharges considered in this study. USGS collected suspended load samples at the Ross gage during water years 1978-1980. The samples were analyzed to determine particle size distributions. Twenty-three events were reported with water discharges varying between 47 and 1,560 cfs. The highest measured flow was well below the design flood peak of 6,900 cfs. In addition to the suspended load measurements, seven bed-load samples were collected during water year 1978 at the Ross gage. Particle size distributions of these samples were also reported. Water discharges when the samples were collected varied between 47 and 1,180 cfs. The measured data were insufficient to determine a reliable sediment inflow rating curve for the model.

30. An initial sediment inflow rating curve was developed based on an optical fit of measured data for water discharges less than 2,000 cfs and on equilibrium transport for higher discharges. Equilibrium transport was calculated using the SEDTRAN transport function (described in the section on transport function) and a bed material gradation determined from four samples collected in 1985 by the Sacramento District. These samples were collected in the channel reach between 750 ft downstream and 250 ft upstream of the Ross gage.

31. The sediment inflow rating curves were adjusted during the adjustment phase of the study. In general, the concentrations of sands were increased, but the concentrations of coarse gravels were decreased. Sediment inflow within the range of sampled data was generally unchanged. Calculated transport of coarse and very coarse gravel did not agree with sampled transport; therefore, sediment inflow concentrations for these coarsest particles were based on equilibrium calculations in the numerical model. The justification for the adapted sediment inflow rating curves is based on successful

simulation of measured deposition in the concrete channel as discussed in Part III, "Adjustment and Circumstantiation." Adapted sediment inflow rating curves are shown in Plates 7-15.

Transport Function

32. A modification of the Laursen equation (Laursen 1958) was developed for use in this study. The Laursen function is desirable because it was developed for size class analysis and considers parameters essential to both bed and suspended sediment loads. The modified Laursen equation (labeled SEDTRAN) incorporated data for transport of gravels in addition to the sand data used to develop the original Laursen function.

33. The SEDTRAN function calculates the hydraulic radius due to grain roughness using the Limerinos equation. This value (instead of the depth as proposed by Laursen) is then used to calculate the grain shear stress:

$$\tau'_o = \frac{\rho V^2}{58} \left(\frac{D_{50}}{R'_b} \right)^{1/3} \quad (4)$$

where

τ'_o = grain shear stress

ρ = water density

V = water velocity

This equation is dimensionally homogeneous and can be applied with any consistent set of units.

34. Bed-load transport is a function of the ratio of applied to critical shear stress. In the SEDTRAN function this is expressed by the parameter

$$(\tau'_o / \tau_{ci}) - 1 \quad (5)$$

where τ_{ci} is the critical shear stress for the i^{th} grain size. The critical shear stress varies with particle size, larger particles having greater critical shear stress. Paintal (1971) determined that the critical shear stress, as used in Equation 5 to determine sediment transport, also varied with applied shear stress. When the dimensionless shear stress (τ'_o) was less than

0.05, he found that the critical shear stress decreased significantly:

$$\tau_o^* = \frac{\tau_o}{(\gamma_s - \gamma_w) D_i} \quad (6)$$

where

τ_o = applied shear stress

D_i = geometric mean diameter of the i^{th} size class

This variation in critical shear stress is accounted for in SEDTRAN by varying the Shields parameter between 0.039 and 0.020. The higher value, recommended by Laursen (1958), was used when τ_o^* was greater than 0.05. The lower limit was determined by Andrews (1983). The effect of this change is that initiation of motion for coarser particles occurs at lower shear stresses, and the transport potential of coarser particles is increased.

35. The SEDTRAN function uses the ratio of grain shear velocity (instead of total shear velocity) to grain fall velocity as the important parameter influencing suspended sediment transport. A functional relationship between this ratio and other parameters was determined by Laursen (1958) based on river and flume data. Due to the reformulation of Laursen's parameters, a new functional relationship was developed for SEDTRAN. The relationship is based on data from both rivers and flumes. The functional relationship and data scatter are shown in Plate 16. Flume data gathered under more controlled conditions have significantly less scatter than the river data.

36. Sediment concentration is calculated by SEDTRAN using the following formula:

$$C = 0.01 \gamma_w \sum_i P_i \left(\frac{D_i}{y} \right)^{7/6} \left(\frac{\tau_o'}{\tau_{ci}} - 1 \right) f \left(\frac{\eta_i'}{\omega_i} \right) \quad (7)$$

where

C = concentration in weight per unit volume

P_i = fraction of grain size class in the bed

y = water depth

τ_o' = bed grain shear stress

U'_* = grain shear velocity

ω_i = fall velocity

$f\left(\frac{U'_*}{\omega_i}\right)$ = function defined in Plate 16

This function is considered to be more theoretically correct than Laursen's original equation and is based on a wider range of physical data. The primary benefit is that it moves coarser gravels better than other functions. However, for the coarsest gravels (greater than 16 mm) the function still does not transport as much material as was sampled in Corte Madera Creek.

PART III: MODEL ADJUSTMENT AND CIRCUMSTANTIATION

Adjustment to Deposition Surveys

37. The historical hydrograph between October 1972 and July 1982 was simulated with the numerical model. This represents the time period between completion of the existing concrete channel and the first available survey of channel deposition. The survey was completed the summer after the flood of record on Corte Madera Creek. Average depths were determined from the surveyed deposition profile in the V-shaped bottom portion of the channel and an average-depth profile was developed for comparison with calculated profiles from the numerical model. Surveyed and calculated deposition profiles are compared in Figure 4. The numerical model overestimated deposition in the mild-sloped reach of the concrete channel, but reproduced both the depth and longitudinal extent of deposition in the steep-sloped portion of the channel.

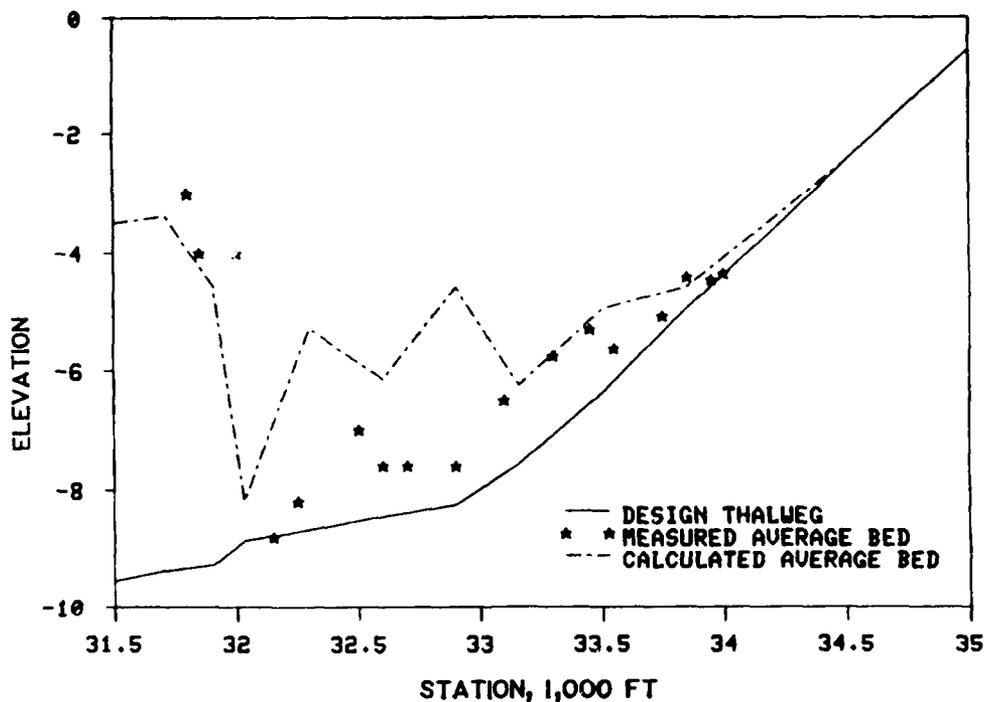


Figure 4. Aggradation in concrete channel, October 1972 to April 1982

38. The hydrograph between October 1972 and August 1984 was simulated with the numerical model. Another deposition survey was taken and bed material samples were collected in August 1984. The time period between the first survey, which was taken in July 1982, and August 1984 was a relatively high

runoff period (Plate 2). The runoff hydrograph at the Ross gage peaked at 2,690 cfs. The calculated and surveyed deposition profiles are compared in Figure 5. The model satisfactorily simulated deposition in the mild-sloped

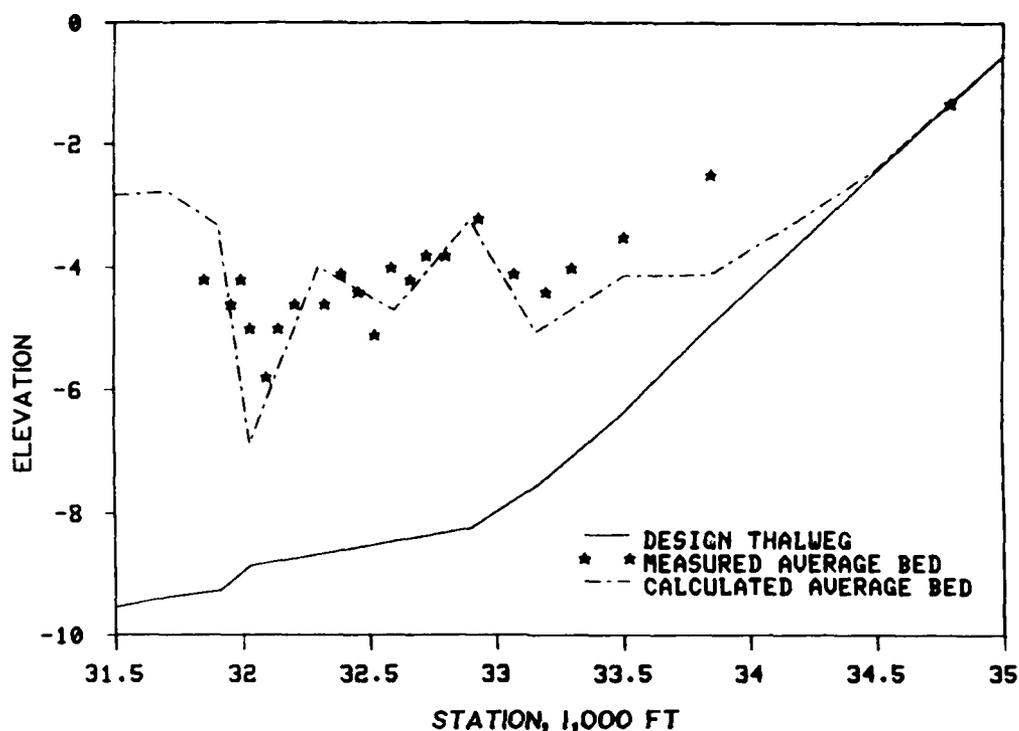


Figure 5. Aggradation in concrete channel, October 1972 to August 1984

portion of the channel, but underestimated deposition in the steep-sloped portion of the channel. During this 2-year period, both the calculated and surveyed profiles showed that the depth of deposition increased in the concrete channel. Bed material samples were collected at sta 335+06, which is the College Avenue Bridge, and at sta 341+00, which is 250 ft upstream of the College of Marin Bridge. Calculated gradations at model cross sections in this reach were compared to the average gradation of these two samples. The sample gradations were found to be considerably finer than the calculated gradations. It was determined from sensitivity studies (discussed in a subsequent section) that an increase in sediment inflow would result in an increase in the coarseness of the calculated gradation and that a decrease in sediment inflow would result in a finer calculated gradation. In order to improve the comparison between measured and calculated gradations, the sediment inflow into the model was adjusted. Gravel inflow was decreased and inflow of fine sands increased. The adjusted sediment inflow rating curves (Plates 7-15) improved the

comparison between sampled and calculated bed material gradations, but the calculated gradation was still coarser. Sampled and calculated bed material gradations for August 1984 are shown in Figures 6a and 6b, respectively. Further reduction in the gravel inflow was not deemed appropriate because it would deviate too much from the measured sediment inflow data and because it would reduce the correlation between measured and calculated depths of deposition.

Model Circumstantiation

39. At this point in the study the numerical model was considered to be adequately adjusted for the prediction of general deposition patterns in Corte Madera Creek. Circumstantiation of the model was accomplished by continuing the historical simulation to January 1986, when another deposition survey was taken. The period between August 1984 and January 1986 had relatively high runoff with the largest peak being 2,600 cfs. Surveyed and calculated deposition profiles are compared in Figure 7. Accumulated deposition volumes are compared in Figure 8. The January 1986 deposition profile (Figure 7) shows little change from the August 1984 deposition profile (Figure 5). The model continued to reproduce an accurate profile downstream of the College Avenue Bridge, but underestimated deposition upstream. The model was very successful in predicting the quantity of total deposition in the channel.

40. A significant runoff event occurred in February 1986, when an estimated peak discharge of 4,150 cfs occurred at the Ross gage. Bed material samples were collected at three locations in the concrete channel upstream from the College of Marin Bridge in March 1986. A deposition survey in the concrete channel was taken in May 1986. Bed material samples were collected at 15 locations between sta 326+00 and 337+50 in May 1987, and three samples were taken laterally across the channel at sta 337+00 in September 1987. The period between May 1986 and May and September 1987 had relatively low runoff, with a maximum peak discharge of 2,500 cfs and only 5 days when the average daily flow exceeded 100 cfs. Due to the small amount of runoff between May 1986 and the collection of the bed material samples in 1987, it was deemed reasonable to compare these measurements with calculated gradations.

41. Calculated and surveyed deposition profiles for the October 1972 - May 1986 simulation are compared in Figure 9. Changes in accumulated

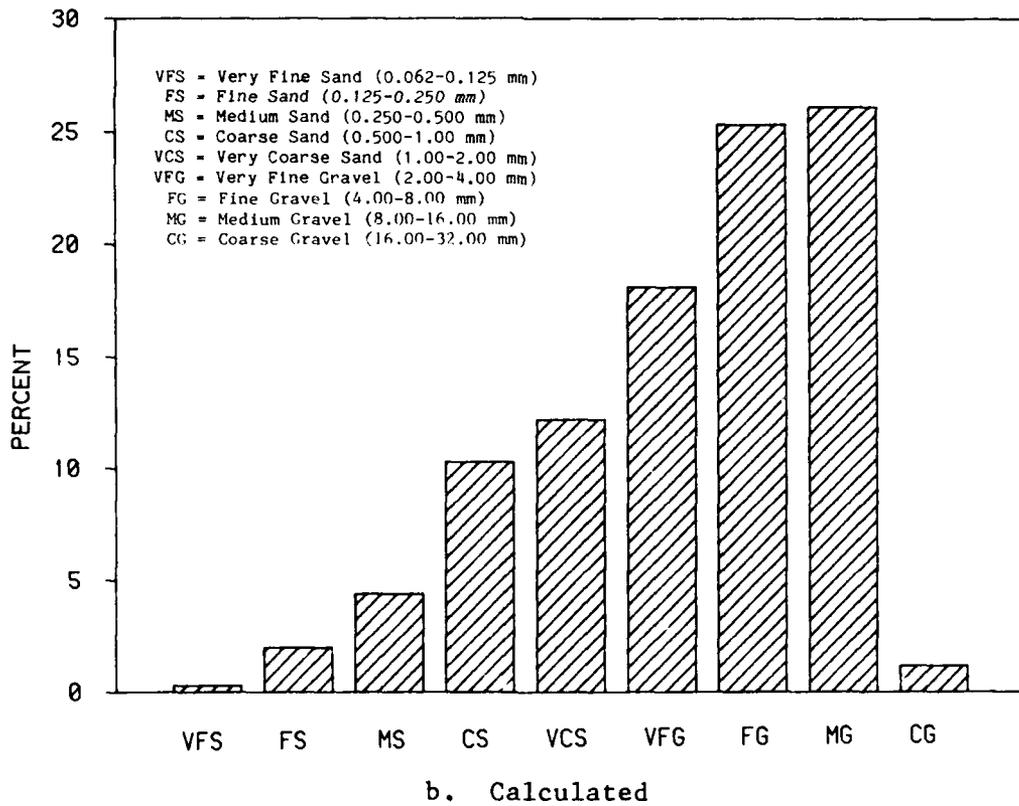
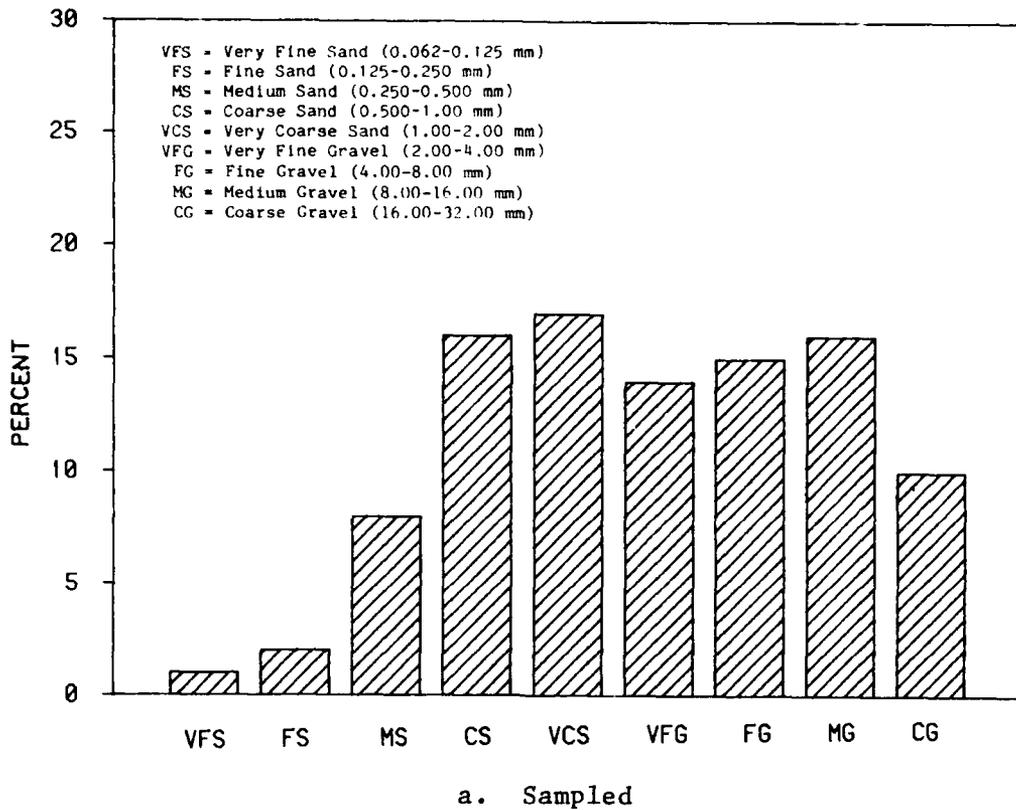


Figure 6. Bed material gradations in the vicinity of College Avenue, August 1984

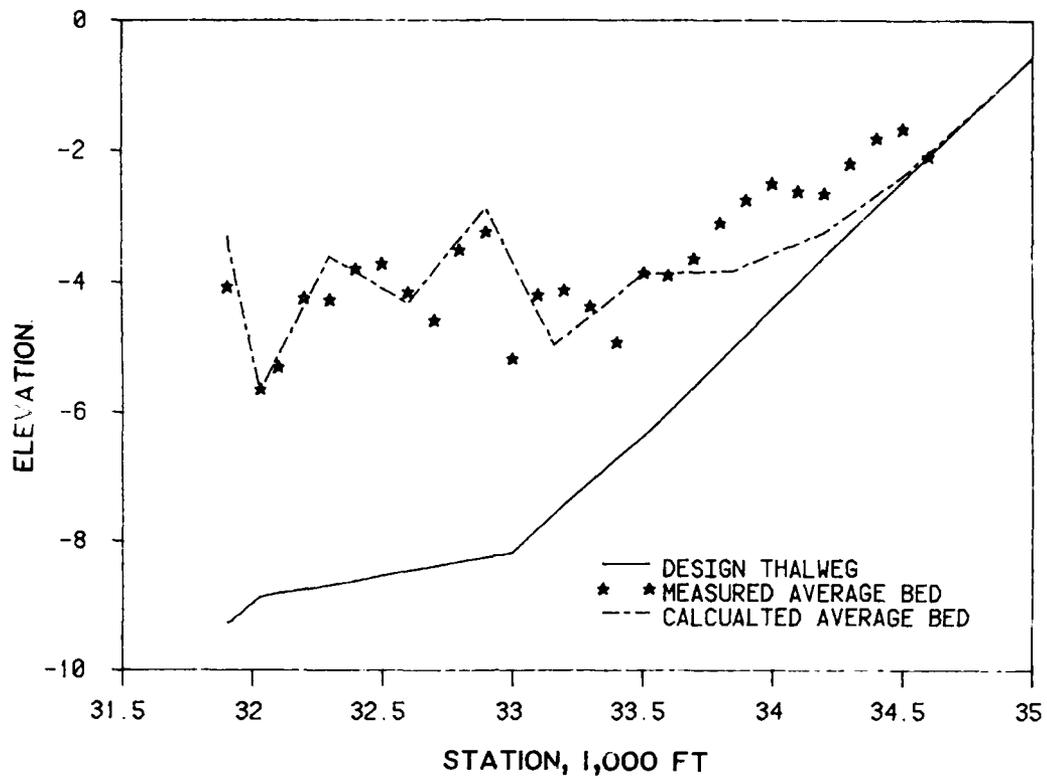


Figure 7. Aggradation in concrete channel, October 1972 to January 1986

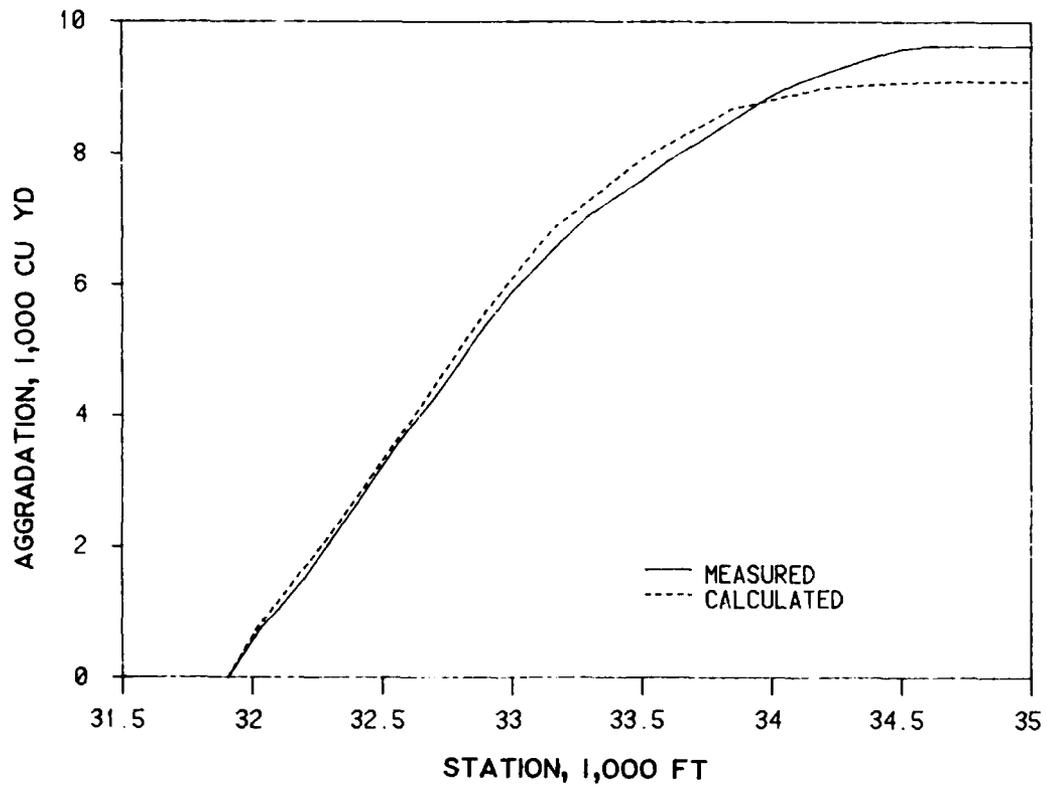


Figure 8. Accumulated aggradation in concrete channel, October 1972 to January 1986

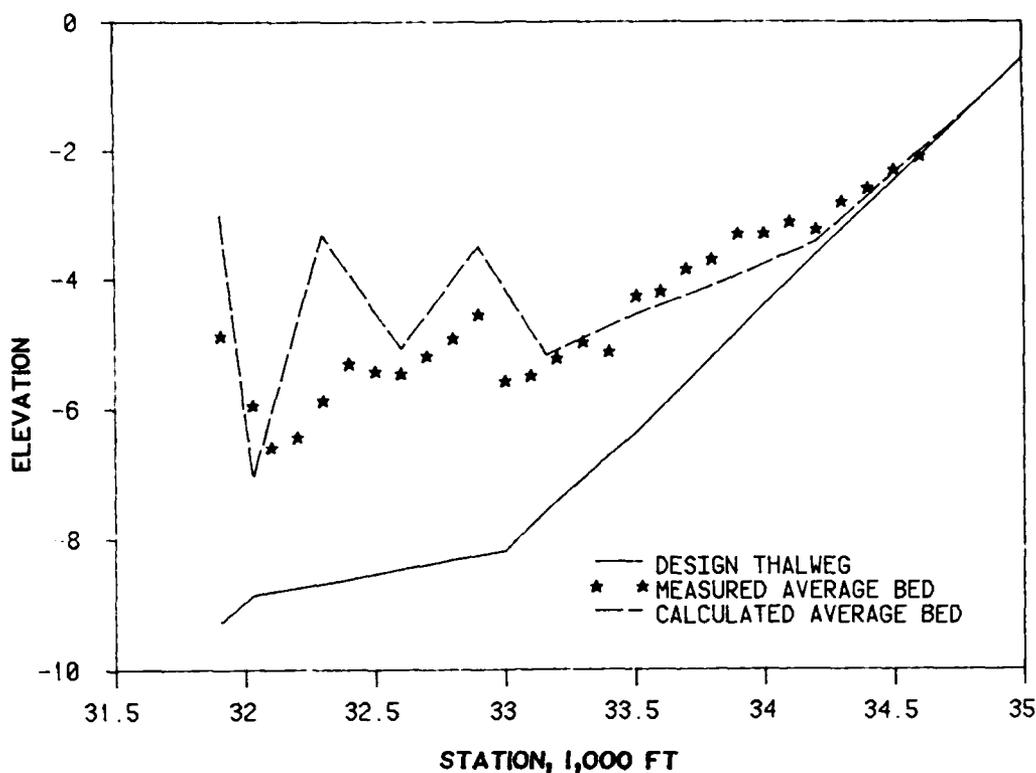


Figure 9. Aggradation in concrete channel, October 1972 to May 1986

deposition in the concrete channel are shown in Figure 10. Based on field surveys, 2,800 cu yd of material were removed from the concrete channel during the February 1986 flood. This compares with a removal of about 1,200 cu yd calculated in the model. The result is consistent with the comparison of field surveys and calculated deposition profiles after the flood of January 1982, which also showed more material removed in the prototype. These results indicate that the model, using average sediment inflow rating curves, underestimated the ability of the concrete channel deposits to degrade during floods. This behavior in the model may be attributed to one or more of the following factors:

- a. The sensitivity study demonstrated the importance of sediment inflow concentrations on the resultant deposition. It is possible that sediment inflow concentrations during the two flood events were significantly different from the long-term averages developed from the measured data. The measured sediment inflow data were taken between 1978 and 1980, which were fairly normal runoff years with a maximum peak discharge of 2,910 cfs.
- b. Model roughness in the concrete channel may be too high. The high roughness values used in the model were chosen in order to match high-water marks from the January 1982 flood. These high roughness values reduce the velocity and thus the scouring

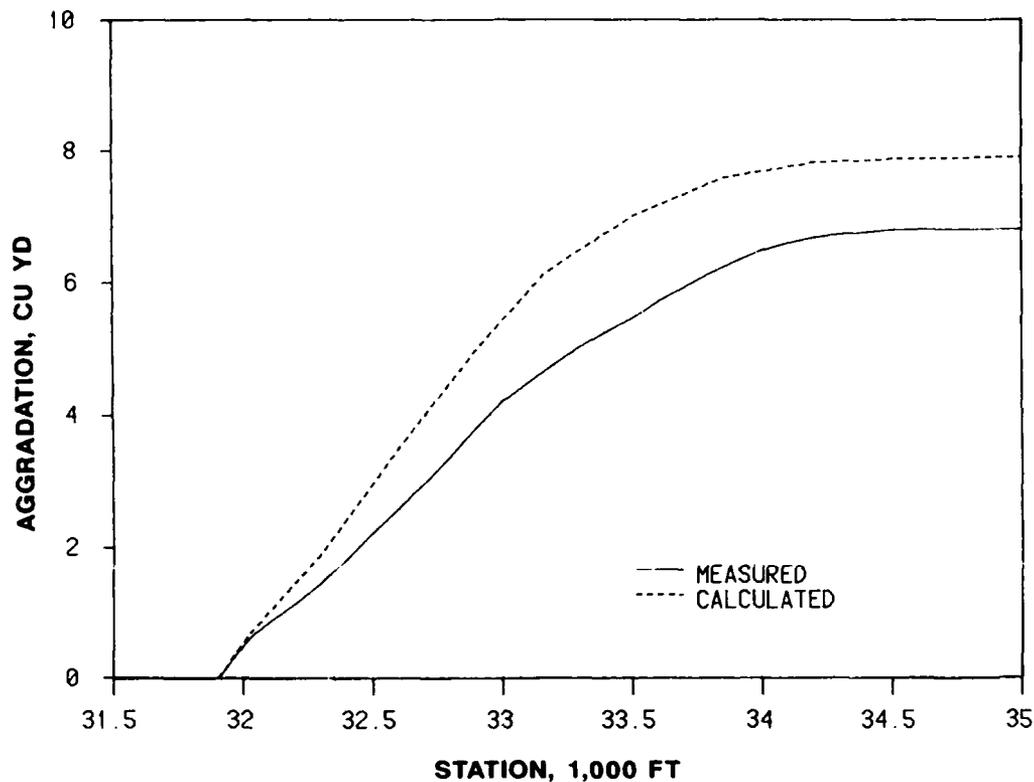


Figure 10. Accumulated aggradation in concrete channel, October 1972 to May 1986

potential of the flow. It is possible that the high-water marks were the result of wave crests and not average water-surface elevations. Significant wave action is typical in channels flowing near critical depth such as Corte Madera Creek during flood events.

- c. Flow breaks away from the Corte Madera channel upstream from the concrete channel during flood flows. If the flow remaining in the channel is underestimated, then the model will show a reduced scouring potential.

42. Bed material gradations from samples taken in 1984-1987 are compared in Figure 11. Lateral and longitudinal variations in bed material gradations are not unusual in gravel bed streams. Therefore, average bed material gradations determined from a large sample population will be more reliable. The March 1986 gradation is finer than the other gradations. These data came from three samples between sta 340+50 and 321+00. One sample compared favorably with 1987 data; the other two had bimodal distributions. Ten samples between sta 326+00 and 337+50 were used to obtain the May 1987 gradation. Three samples, varied laterally at sta 336+80, were used to obtain the September 1987 gradation. Bed gradations calculated at the end of the 14-year

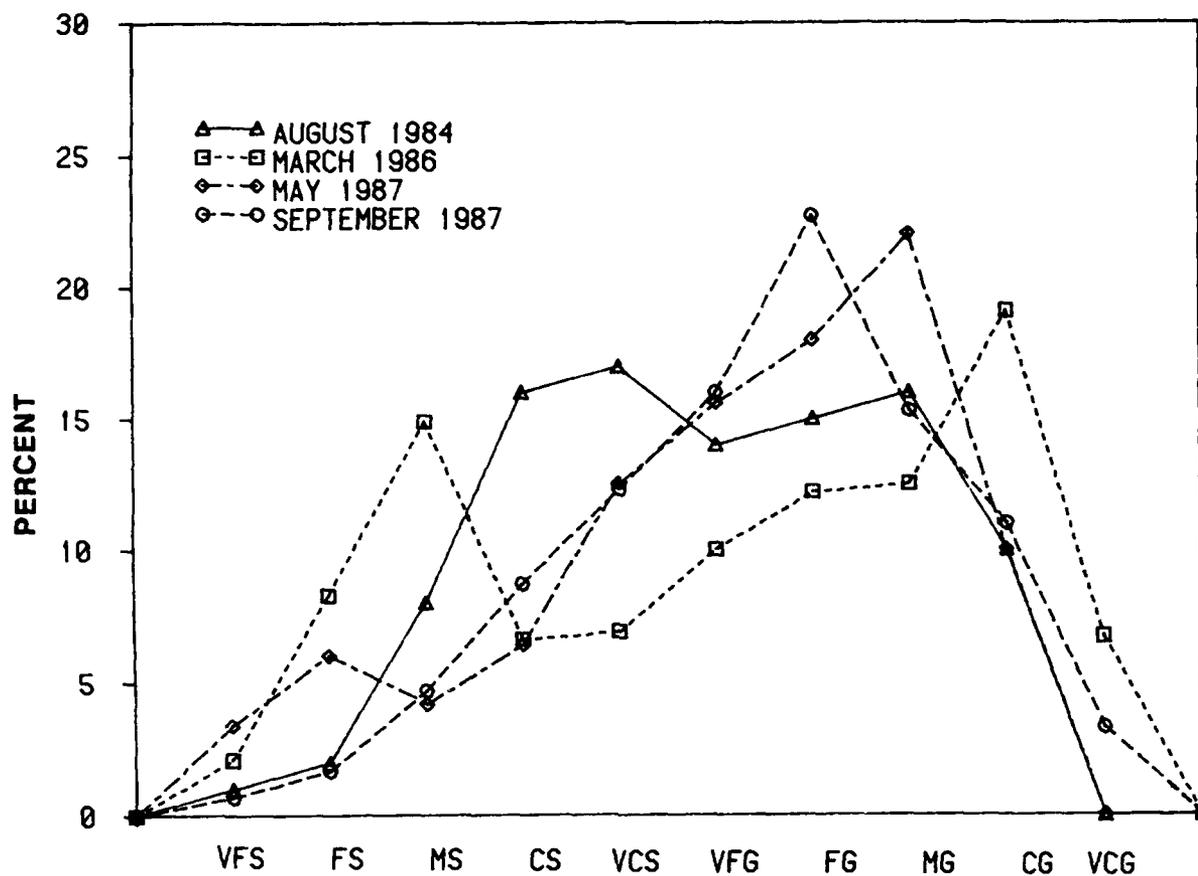
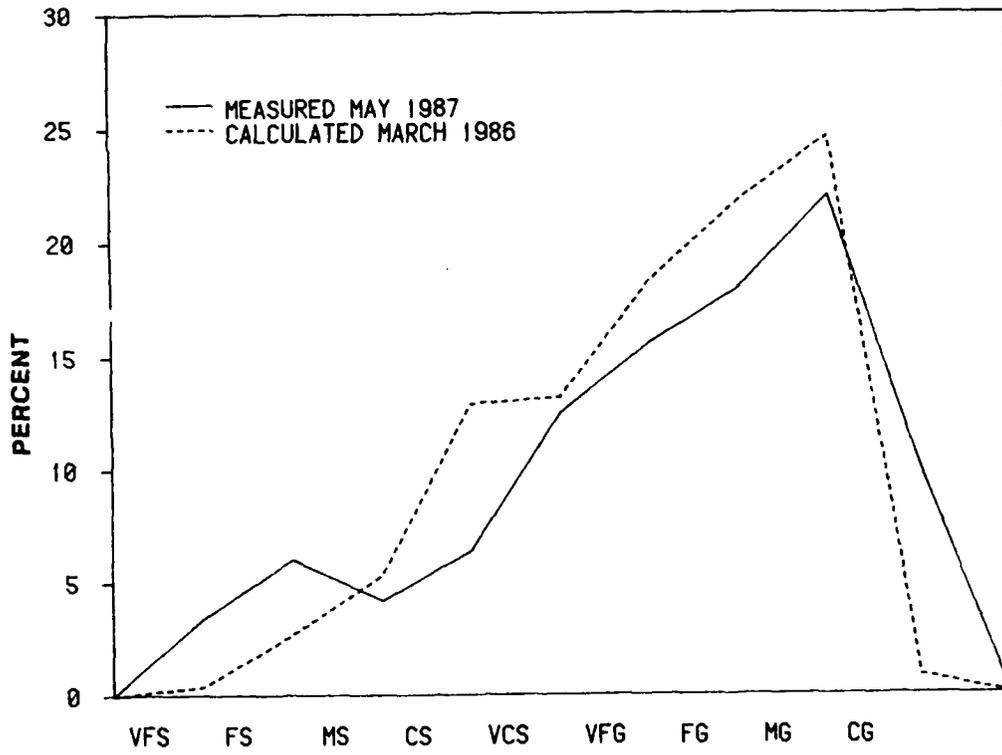


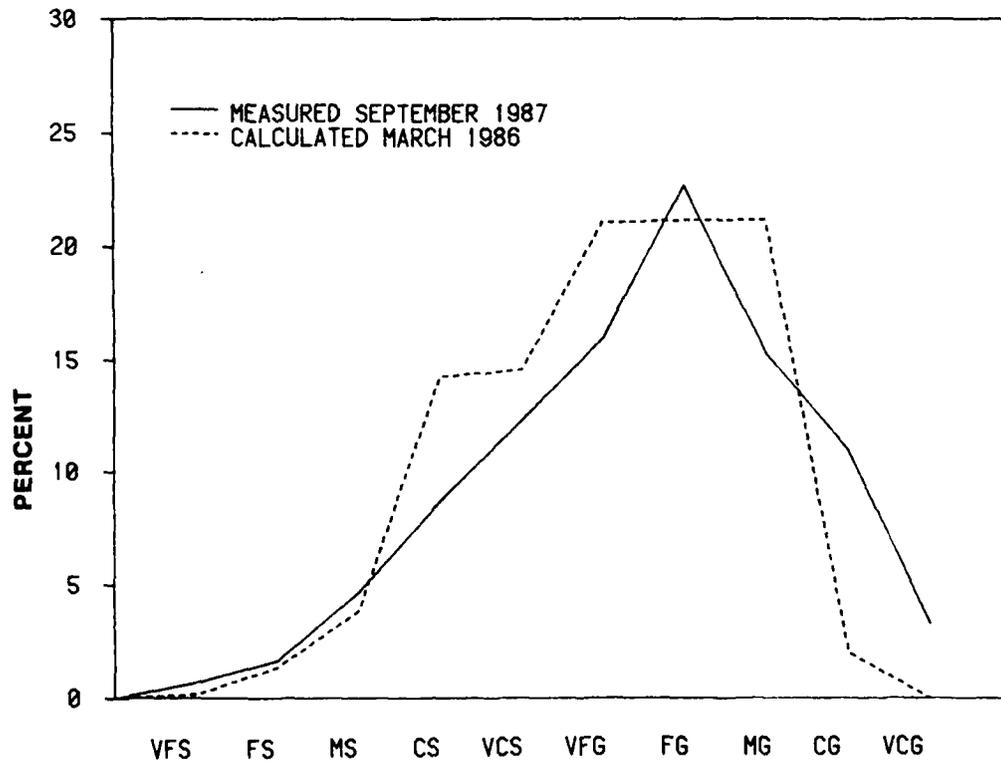
Figure 11. Sampled bed material gradations

simulation are compared to the 1987 sample gradations in Figure 12. This comparison is much better than the comparison of measured and calculated gradations in 1984. The calculated bed material gradations are well within the scatter of prototype data, and model performance can be considered reliable.

43. The numerical model can be used to evaluate the proposed channel improvements for Unit 4 on Corte Madera Creek. It is recognized that the reliability of model predictions is somewhat limited due to the uncertainty related to prototype sediment inflow concentration and variations in bed material gradation data. The model was successful in simulating the longitudinal extent and general quantity of deposition in the concrete channel. Bed material gradations were reproduced fairly accurately. These gradations are important because they influence the roughness of the channel bed. The model predicted degradation during flood events. Because predicted quantities of degradation were less than the measured quantities, the model will provide



a. Between stilling basin and College of Marin bridge



b. Between College Avenue and College of Marin bridges

Figure 12. Bed material gradation

conservative results with respect to the ability of the channel to maintain a sediment-free bed with the addition of the sediment basin in Unit 4.

Sensitivity Study

44. The sediment inflow rating curves used in the adjusted numerical model were based on suspended and bed-load samples taken at water discharges less than 2,000 cfs and on equilibrium transport calculations, with some adjustment to better simulate deposition surveys and sampled bed material gradations. Due to the critical importance of sediment inflow on deposition in the concrete channel, the sensitivity of the adjusted numerical model to sediment inflow was tested. The average values for sediment inflow were increased by a factor of 1.5 and decreased by a factor of 0.5 in the sensitivity study. As shown in Plates 7-15, these values are still within the range of measured data. Results of a historical simulation from October 1972 to August 1984 are compared in Figure 13. As expected, the sediment inflow rating curve influences the deposition profile in the concrete channel. The uncertainty and significance of sediment inflow must be considered when interpreting study results.

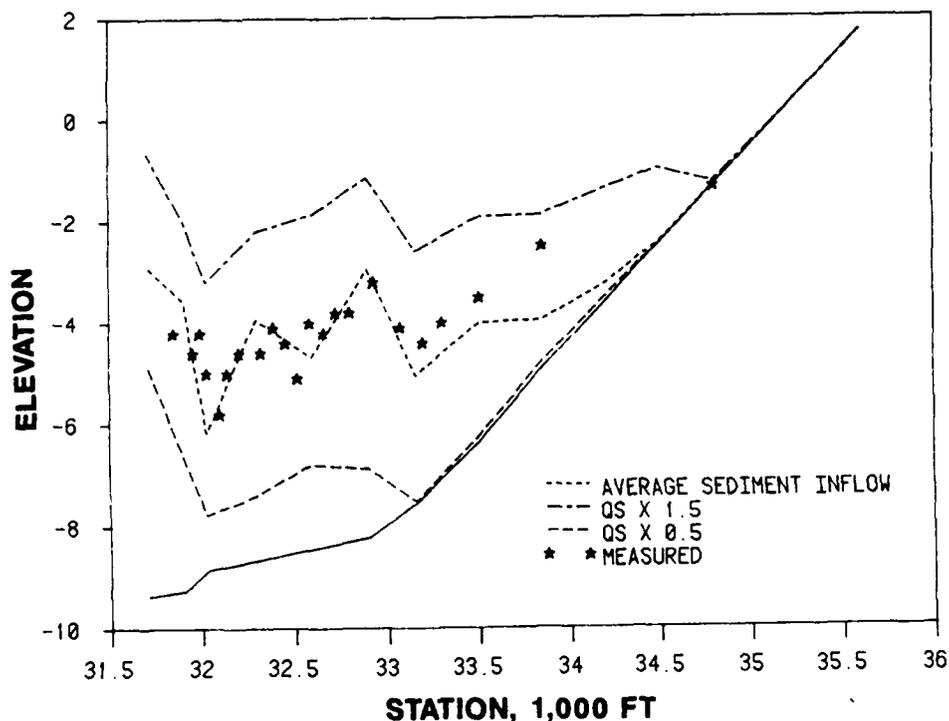


Figure 13. Sensitivity of model to sediment inflow, measured and calculated aggradation, August 1984

PART IV: STUDY RESULTS

Standard Project Flood

45. The proposed design for Unit 4 (Plate 1) on Corte Madera Creek was incorporated into the numerical model, and the SPF histogram (with no antecedent runoff) was run. The initial channel bed elevations were assumed to be at design levels. Due to the rapid change in cross-section shape at the sediment trap, the numerical integration scheme in the model was set to use "at-station" values for hydraulic parameters, rather than the average values used in the historical simulation. Although the use of at-station values can decrease model stability, especially for long-term simulations, the time-steps used for flood simulations were short enough to prevent model instability problems. At the SPF peak of 7,500 cfs, about 2,600 cu yd of material had deposited in the sediment trap. The concrete channel was essentially free of gravel and sand on the rising limb of the flood hydrograph and at the flood peak. On the recession limb, when the discharge became less than 3,000 cfs, gravel and sand began to deposit in the concrete channel. Shear stresses were high enough to allow both sand and gravel to pass through the sediment trap during the flood peak and on the recession limb of the hydrograph. By the end of the flood, 3,100 cu yd of material were deposited in the sediment trap and 1,800 cu yd in the concrete channel. Calculated gradations of deposited material are shown in Figure 14. These gradations show that the sediment trap was effective in trapping both sand and gravel sizes, and that sufficient quantities of coarse material deposited in the concrete channel on the flood recession, causing the high bed roughness characteristic of movable gravel beds. This will create hydraulic efficiency problems if another flood occurs before the deposit is removed.

46. The channel reach downstream of the sediment trap has an earthen bottom for about 200 ft. When the sediment trap operates effectively, sediment concentrations in this reach will be less than the possible sediment concentration. As a result, there is a potential for degradation of the channel and scour at the toe of the bank protection. A test using the numerical model evaluated this potential for the SPF by removing the limits to degradation that were imposed during the adjustment of the model. This procedure is not completely valid because the paved channel bottom was a part of the model

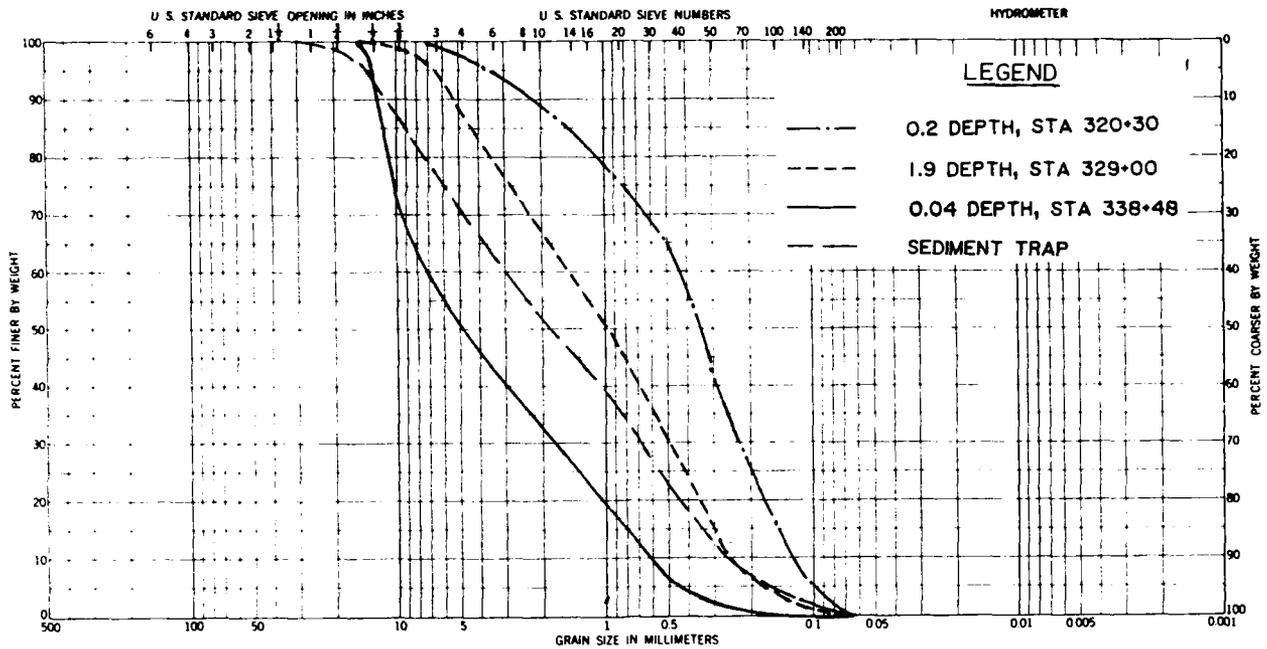


Figure 14. Bed material gradations at end of SPF

adjustment procedure. Therefore, results should be considered only in a qualitative manner. Model results showed a maximum of 2 ft of degradation in this reach, which indicates that the channel invert in this reach should be protected. Deposition in the concrete channel was not significantly affected, because most of the eroded material was transported through the concrete channel reach. A test was made to determine if decreasing the bed slope to 0.0 in the earthen channel would eliminate degradation in this reach. This change reduced the maximum degradation to 1.8 ft.

Average Annual Deposition

47. An annual flow duration histogram was used to determine average annual deposition in the proposed sediment trap and concrete channel. The annual flow duration histogram was based on flow duration data from 32 years of record, adjusted to include the major flood peaks in 1955, 1982, and 1983. An average annual sand and gravel inflow of 8,200 cu yd was calculated. Calculated average annual deposition in the sediment trap was about 2,000 cu yd. This material was coarser ($D_{50} = 3$ mm) than calculated for the SPF because the SPF had a greater runoff volume and as the sediment trap filled, the trap efficiency for coarse material decreased faster than for fine material. There

was no deposition in the concrete channel until the discharge became less than 2,500 cfs on the recession limb of the hydrograph. Average annual deposition in the concrete channel was about 1,000 cu yd. This material was considerably finer ($0.2 \text{ mm} < D_{50} < 0.5 \text{ mm}$) than the material deposited during the SPF. This size material can be expected to scour out of the channel during the rising limb of the next flood hydrograph.

Initial Deposition in Earthen Channel

48. In the SPF and annual flow duration histogram numerical model tests, the entire flood-control project, including the earthen and concrete channels, had been tested with the initial bed elevation at the design invert. Sacramento District determined that this condition, which would require annual dredging of the entire project, is too extreme for the earthen channel downstream from the stilling basin. Therefore, the effect of initial deposition in the earthen channel on water-surface elevations in the concrete channel was evaluated. An acceptable maximum level of initial deposition in the downstream channel was determined and established as a criterion for maintenance dredging. The 1984 channel survey was used to determine longitudinal distribution of sediment deposits. Initial depositions ranging from 0.0 (design invert) to 1.6 times the 1984 deposition depths were tested in the model. Tested thalwegs are shown in Plate 17.

49. The effect of initial deposition in the earthen channel on water surfaces and deposition in the concrete channel was tested with the design flood (100-year frequency) histogram, which simulated the 1982 flood season with the design flood in place of the 4-5 January flood. Calculated peak water-surface elevations from the numerical model are shown in Plate 18. Initial channel deposition downstream from the stilling basin resulted in increases in water-surface elevations in the concrete channel. When the initial deposition in the earthen channel was 1.4 times the 1984 deposition or less, sediment that deposited in the concrete channel prior to the flood peak was removed by the time the peak discharge occurred. The roughness algorithm used by the model calculated a composite channel roughness coefficient of 0.022 in the concrete channel below sta 342+00. When initial deposition was 1.6 times the 1984 deposition, sediment deposited prior to the peak was not completely removed and a higher roughness coefficient of 0.028 was calculated

at some cross sections. A rating curve showing maximum calculated water-surface elevations at three stations as a function of initial deposition in the earthen channel is shown in Plate 19. This plate shows that at the peak of the design flood, with an initial deposition in the earthen channel the same as the 1984 survey, calculated water-surface elevations would be 1.3, 2.3, and 3.9 ft above the existing top of wall at sta 326+00, 335+06, and 342+00, respectively. It was determined, in consultation with Sacramento District, that deposition depths 0.4 times the 1984 deposition should be assumed in the earthen channel for evaluation of design conditions in the concrete-lined channel. The results of this study would then be used by the Sacramento District for determining the actual design maintenance criteria required in the downstream earthen channel.

Design Conditions: Base Test

50. The following design conditions were used for testing the sedimentation effects in the proposed channel: (a) concrete channel and sediment trap maintained to design invert elevations at the start of the flood season, and the earthen channel bottom elevations maintained at a level not to exceed 0.4 times the 1984 sediment deposition elevations (Plate 17); (b) the 1982 flood season, with the 100-year frequency flood replacing the 4-5 January flood, for a design histogram; (c) roughness coefficients in the concrete channel determined from the January 1982 simulation; and (d) sediment inflow based on long-term average inflow determined by reproducing measured deposition in the channel.

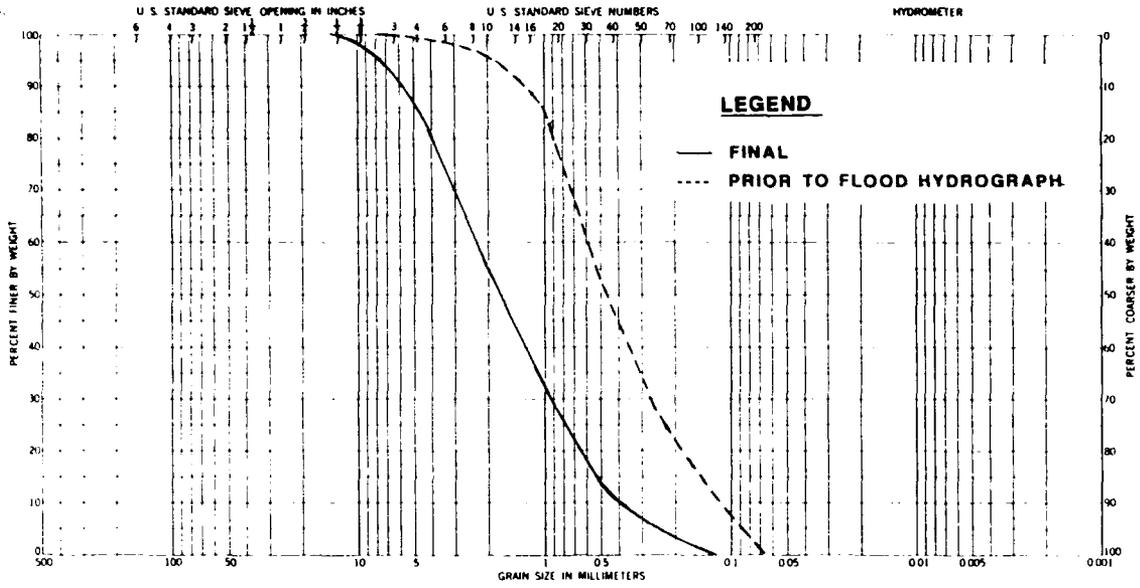
51. The design channel operated satisfactorily in the numerical model test with the design histogram. About 900 cu yd of sand accumulated in the concrete channel during the antecedent flow period. The sediment trap had accumulated about 2,000 cu yd of material just before the start of the flood histogram. At the peak of the flood, 3,900 cu yd were stored in the sediment trap, 60 percent of which was sand and 40 percent gravel. Most of the sediment that had deposited in the concrete channel prior to the flood peak had washed out by the time the discharge reached 5,000 cfs. Calculated deposition in the concrete channel at the peak was less than 0.05 ft and can be considered to represent bed load. The D_{84} of this material was less than 2 mm, so no increase in roughness was calculated by the model's roughness algorithm.

Maximum calculated water-surface elevations for the design flood were below the existing wall between the stilling basin and sta 328+00, about 1 ft higher than the existing wall at College Avenue, and about 2.4 ft higher than the existing wall at sta 342+00. The TABS-1 model does not account for bridge losses; therefore, design water-surface elevations will be higher. HEC-2 backwater calculations by Sacramento District indicated that about 0.7 ft of head loss can be expected by the combined College Avenue and College of Marin bridges. Flows subsequent to the flood peak deposited material in the concrete channel. Some of this material came from the sediment trap. At the end of the flood season, 2,800 cu yd of sediment were deposited in the trap and 4,500 cu yd in the concrete channel. The channel deposits at the end of the season were considerably coarser ($D_{84} = 5$ mm) than deposits from the antecedent flow ($D_{84} = 1$ mm). The material deposited in the trap at the end of the season was also coarser ($D_{84} = 12$ mm) than deposits at the peak and from the antecedent flow ($D_{84} = 6$ mm). Calculated gradations of the channel and trap deposits are shown in Figure 15. Bed changes during the passage of the test hydrograph are shown for three concrete channel sections in Figure 16.

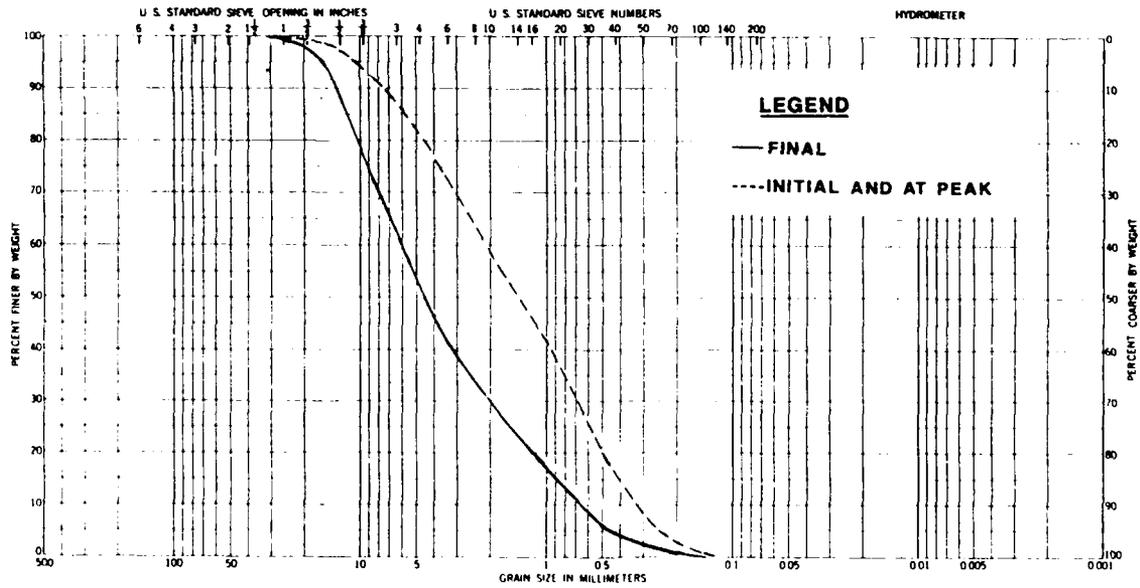
Sensitivity Studies

Roughness coefficient

52. Sensitivity studies were conducted with the design histogram to determine the effect of the roughness coefficient on water-surface elevations and deposition in the lower portion of the concrete channel. With the base test, the calculated roughness coefficient was 0.022. A high roughness coefficient (0.031), representing high form roughness due to gravel movement, and a low roughness coefficient (0.017), representing a channel with some abrasion but free from deposits, were tested. Calculated water-surface elevations are compared in Figure 17 and Table 1. As expected, channel roughness had a significant effect on water-surface elevations, emphasizing the importance of removing gravel deposits on an annual basis. A maximum increase in water-surface elevation of 2.9 ft above the base condition was calculated with the high roughness coefficient. A maximum decrease of 2.5 ft below the base condition was obtained with the low roughness coefficient. Interestingly, even with the higher water-surface elevations and accompanying decrease in velocities calculated using the high roughness coefficient, the antecedent



a. Sta 329+00



b. Sediment trap

Figure 15. Bed material gradations

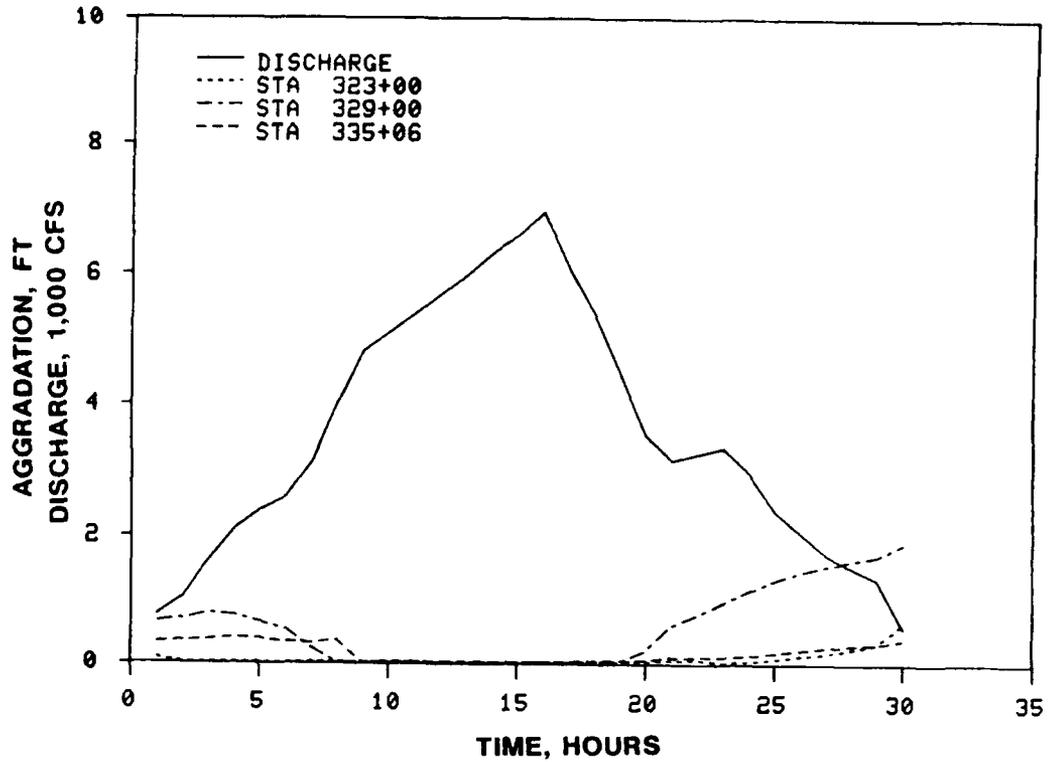


Figure 16. Aggradation in concrete channel with passage of design flood hydrograph

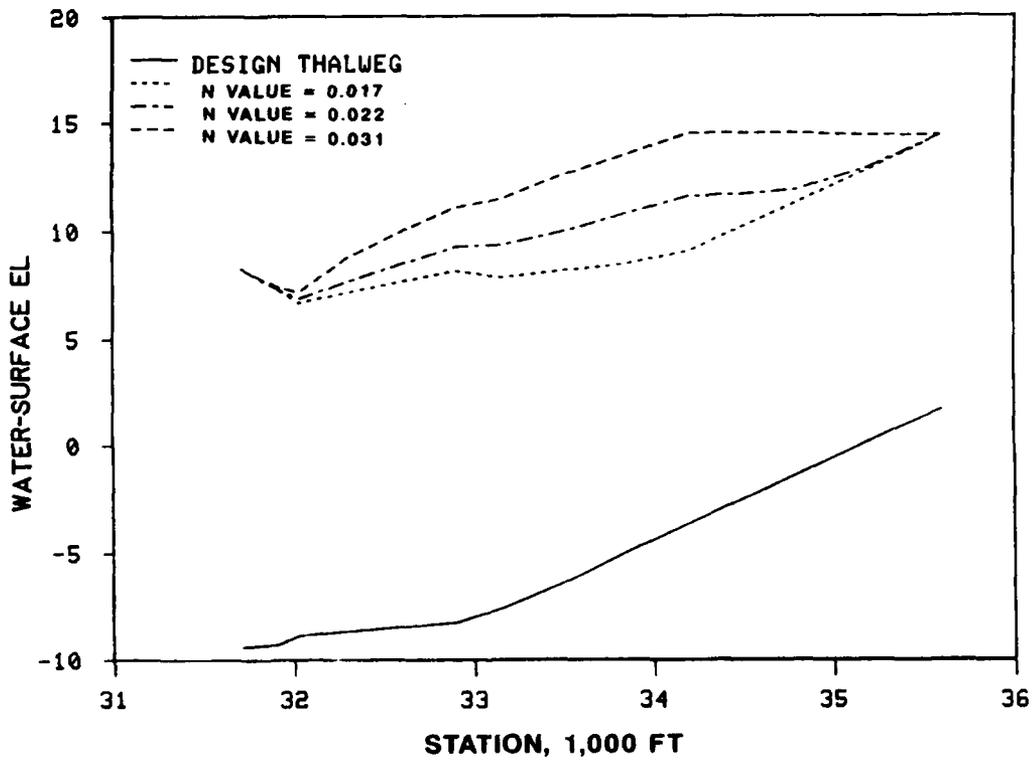


Figure 17. Sensitivity to roughness coefficient at design flood peak

deposition was washed out of the concrete channel prior to the flood peak.

Downstream water-surface elevation

53. The effect of the downstream water-surface elevation was tested. Design simulations had been run with a downstream water surface at mean sea level (0.5 ft NGVD). A downstream water-surface elevation of 3.24 ft NGVD, which was the mean higher high water elevation for the January 1982 period, was tested and resulted in about a 0.5-ft increase in calculated water-surface elevations in the lower portion of the concrete channel. Calculated values are shown in Table 1. There was no change in calculated roughness with the higher water surface elevations.

Sediment inflow

54. The sensitivity of the proposed design to sediment inflow was tested with the design histogram. The average values for sediment inflow were increased by a factor of 1.5 in the sensitivity test. There was more deposition in the concrete channel during antecedent flow when the sediment inflow was increased. However, these deposits were primarily sand and were washed out of the concrete channel prior to the flood peak. There was no increase in the calculated roughness coefficient at the flood peak. Water-surface elevations at the stilling basin were about 0.8 ft higher (Table 1). This increase is attributed to greater deposition and the subsequent decrease in channel conveyance in the downstream earthen channel.

Transport function

55. The sensitivity of model results to the transport function was tested using the Unit Stream Power equations of Yang (Yang 1973, 1984), which have been widely applied. The Yang transport equations did not move as much gravel as the SEDTRAN function, resulting in deposition at the upstream model boundary. However, in the concrete channel, results were the same with respect to sediment deposition during the peak of the design flood. The channel bottom was free of coarse sediment deposits during the test hydrograph peak discharge, indicating no increase in roughness due to gravel movement.

Antecedent flow

56. The effect of increasing antecedent flow was tested by moving the design flood hydrograph to the end of the 1982 flood season (Plate 20). With this change, the volume of sediment deposited in the concrete channel just prior to the beginning of the flood histogram was about 2,900 cu yd, compared to 900 cu yd with the design histogram. The volume of sediment stored in the

trap was about the same: 2,400 cu yd with the greater antecedent flow compared to 2,000 cu yd with the design histogram. By the time the flood peak arrived, most of the sediment had been washed out of the concrete channel for both flow conditions. However, with the greater antecedent flow, there was sufficient gravel in the calculated bed load to cause an intermittent increase in the calculated roughness coefficient. Calculated peak water-surface elevations, shown in Table 1, at the stilling basin were about 0.4 ft higher with the greater antecedent flow. This increase is attributed to increased downstream deposition due to the increase in runoff volume. The numerical model's roughness algorithm calculated a roughness coefficient of 0.028 at some cross sections with the greater antecedent flow. As a result, peak water-surface elevations were about 1 ft higher in the concrete channel. Calculated sediment thicknesses were less than 0.05 ft in the model, indicating that the entire 0.010 increase in the bed roughness coefficient to account for gravel form roughness may be too severe for this case. The composite roughness coefficient of 0.028 is slightly lower than the average composite value of 0.030 that was calculated for the historical simulation, where the bed material deposit was coarser and thicker.

Effect of Sediment Trap

57. The numerical model geometry was revised by removing the sediment trap from the Unit 4 design. The effectiveness of the sediment trap was determined by running this revised model with the design histogram. The antecedent flows deposited about 1,800 cu yd of sand and gravel in the concrete channel. This is about twice the amount deposited with the sediment trap. With the trap, the D_{84} of the deposited material was in the medium to coarse sand range; without the trap, the D_{84} was in the fine to medium gravel range. At the peak of the design flood, most of the deposited sediment had been washed out for both cases. However, without the trap, there was sufficient gravel in the bed load to cause an intermittent increase in the calculated roughness coefficient to 0.028. Calculated peak water-surface elevations, shown in Table 1, were about 0.3 ft higher at the stilling basin without the trap. As with the greater antecedent flow case, this is attributed to an increase in downstream deposition. Without the trap, the maximum increase in calculated water-surface in the concrete channel was about 1 ft at

sta 342+00. As mentioned in the previous paragraph, sediment deposit thicknesses were small (less than 0.05 ft), and the roughness algorithm may be calculating roughness coefficients that are too severe.

58. An additional 2 ft was dredged inside the sediment trap to determine its effect on gravel deposits in the concrete channel. Since the project performed satisfactorily with the design flood histogram, the deepened trap was tested with the greater antecedent flow histogram. Calculated results were similar to results with the design trap. Bed material gradations were slightly less during the peak flood, and increased roughness due to gravel movement was calculated at only two cross sections compared to four cross sections with the design histogram. As a result, calculated water-surface elevations were slightly less (Table 1). Improvement in performance of the project was not significant enough based on numerical model results to recommend deepening the trap.

Design Roughness Coefficients

59. The calculated composite roughness coefficient in the existing concrete channel between the stilling basin and sta 342+00 at the peak of the design flood was 0.022. Sensitivity studies indicated that minimum freeboard should be determined using a roughness coefficient of 0.028 between the stilling basin and sta 342+00. Backwater analyses were performed by the Sacramento District using these coefficients. These analyses allowed for additional deposition in the earthen channel downstream from the stilling basin, a tide elevation of 2.9 ft, and bridge losses. Calculated water-surface elevations indicated a maximum 5.7 ft of additional wall height would be required in the concrete channel for the design conditions.

60. An analytical technique was employed to determine how much the roughness coefficient would be reduced if the channel wall deposits were removed annually. This technique involved partitioning the calculated composite roughness coefficient to account for contributions of the various roughness elements and then reducing the contribution due to channel wall deposits. The procedure is described in Appendix B. The results of this analysis were a design roughness coefficient of 0.020 downstream from sta 342+00, and a roughness coefficient of 0.026 to determine minimum freeboard.

61. Additional methods to reduce required wall heights would include

increased dredging downstream from the stilling basin, removal of bridges at College Avenue and College of Marin, widening of the concrete channel downstream from College Avenue, and widening and reducing irregularity in the earthen channel downstream from the stilling basin. Upstream from sta 342+00 roughness could be decreased by relining the channel with erosion-resistant concrete.

PART V: CONCLUSIONS AND RECOMMENDATIONS

Conclusions

62. Channel roughness in the concrete channel is affected by gravel deposits. A comparison of backwater calculations and high-water marks from the January 1982 flood demonstrated that the average Manning's roughness coefficient in the concrete channel with gravel deposits was on the order of 0.030 at the flood peak. Sinuosity, grain roughness on the channel bottom, and tubeworm and barnacle growth on the channel walls were insufficient to account for a composite roughness of this magnitude. The additional roughness was attributed to form roughness due to the movable gravel bed. Criteria, which included a minimum thickness and applied shear stress, were established to determine when adding form roughness to the composite channel roughness would be appropriate. A conceptual framework, considered appropriate for making comparative analyses of sedimentation in Corte Madera Creek, was developed to emulate the complicated physical processes as much as possible, given state-of-the-art knowledge. Using this framework, with the design flood conditions, a roughness coefficient of 0.022 was calculated for a channel free from sediment deposits. This value increased to 0.028 when there was sufficient gravel in the movable bed layer, either by increased antecedent flow or deletion of the sediment trap, to cause a calculated increase in roughness. When sediment deposits were not removed prior to the design flood, a value of 0.030 was calculated due to a thicker and coarser deposit.

63. Using average sediment inflow rating curves, the numerical model generally reproduced both measured deposition quantities and sampled gradations over a 14-year historical period. Sensitivity tests demonstrated that sediment inflows of 1.5 and 0.5 times the average would produce deposition considerably different than measured quantities. Therefore, long-term simulations using average sediment inflow are considered fairly reliable.

64. Probable variations in sediment inflow during a flood event make short-term predictions less reliable than long-term predictions. However, the sensitivity tests demonstrated that increasing the average sediment inflow by 50 percent did not result in an increase in calculated roughness coefficient. However, an increase in calculated water-surface elevation of about 0.8 ft occurred due to the greater antecedent deposition in the earthen channel

downstream from the stilling basin. Results from the sensitivity study relieve some possible concern related to sediment inflow uncertainty and its effect on channel roughness.

65. The proposed sediment trap collects both sand and gravel. The numerical model calculated a trap storage of 3,900 cu yd, 60 percent sand and 40 percent gravel, at the peak of the design flood; about 2,000 cu yd of this material was deposited by flow preceding the peak. The effective storage capacity of the trap is not the same as the volume excavated below the existing bed profile, because during floods, sufficient sediment transport potential exists to maintain some sediment movement through the trap. The numerical model results using the design histogram demonstrated that the trap was effective in reducing channel deposition from flow preceding the peak and in reducing the coarseness of both the bed deposit and the bed load at the flood peak. This resulted in a reduction of about 1 ft in the computed water-surface elevation in the concrete channel downstream from sta 342+00. Deposition in the concrete channel at the end of the flood season was about the same, with or without the trap. Deposition in the downstream earthen channel would be increased without the trap.

66. Peak water-surface elevations in the concrete channel downstream from sta 342+00 are directly related to initial deposition in the earthen channel downstream from the stilling basin and the tide elevation. When deposition in the earthen channel was less than 1.4 times the 1984 surveyed deposition (Plate 17), there was no increase in calculated roughness due to gravel deposits in the concrete channel for the design histogram. However, decreased channel conveyance due to deposition in the earthen channel resulted in increases in water-surface elevations in both the earthen and concrete-lined channels.

67. Using the annual flow duration histogram with an average annual sand and gravel inflow of 8,200 cu yd, average annual deposition quantities of 1,900 cu yd in the sediment trap and 1,000 cu yd in the concrete channel were calculated.

68. Degradation of the channel bottom between the sediment trap and the upstream end of the concrete channel can be expected with the proposed design. About 2 ft was calculated for the SPF. This reach can also be a source of material that could increase roughness in the concrete channel.

Recommendations

69. Roughness coefficients downstream from sta 342+00 in the concrete channel were determined primarily from high-water marks and engineering judgment. These coefficients consider the effects of sinuosity and tubeworm and barnacle deposits on the concrete channel walls. Composite roughness coefficients of 0.030 for a channel without annual removal of sediment deposits; and 0.022 for a channel with annual removal of sediment deposits are recommended for design. With the design histogram, sediment deposits were eroded when the discharge exceeded 5,000 cfs. The 0.022 recommendation is contingent on the earthen channel downstream being maintained such that deposition is less than 1.4 times the 1984 deposition (Plate 17).

70. Due to uncertainties related to sediment inflow, antecedent flow, tide elevation, and channel roughness, minimum freeboard should be determined using a roughness coefficient of 0.028. This coefficient was calculated at the peak of the flood during several sensitivity tests, when the deposition thickness was less than 0.05 ft.

71. A maintenance program in the concrete channel that includes annual removal of tubeworm and barnacle deposits from the channel walls (assumed 50 percent effective) in addition to bed deposits would result in a design roughness coefficient of 0.020. Minimum freeboard should be determined using a roughness coefficient of 0.026.

72. With the existing design, it is recommended that the earthen reach between the sediment trap and the concrete channel be protected to prevent degradation.

73. This study considered only the proposed design plan and raising the walls in the concrete-lined channel. Should a plan be formulated with significant modification or change in design discharge, it is recommended that the plan be tested with the numerical model.

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Table 1
Sensitivity of Water-Surface Elevations to Input Parameters

Sta	Base Test*	Elevation, ft NGVD						
		Roughness Coefficients**		Downstream Water-Surface Elevation	Greater Antecedent Flow†	No Sediment Trap	Increased Sediment Inflow	Sediment Trap 2 ft Deeper
		n = 0.017	n = 0.031					
166+40	0.5	Same as base test		3.2	Same as base test		Same as base test	
181+00	1.0	↓	↓	3.4	↓	↓	↓	↓
190+00	1.5	↓	↓	3.7	↓	↓	↓	↓
201+00	2.0	↓	↓	3.9	↓	↓	↓	↓
222+00	2.7	↓	↓	4.3	↓	↓	↓	↓
238+00	3.1	↓	↓	4.6	↓	↓	↓	↓
260+00	4.1	↓	↓	5.1	↓	↓	4.1	↓
280+00	5.1	↓	↓	5.9	5.1	↓	5.2	5.1
293+00	5.9	↓	↓	6.5	6.0	↓	6.0	6.0
303+00	6.7	↓	↓	7.2	6.8	6.7	6.9	6.8
310+00	7.4	↓	↓	7.9	7.6	7.5	7.8	7.5
317+10	8.3	↓	8.3	8.8	8.6	8.5	8.9	8.5
319+05††	7.4	7.4	7.5	7.9	7.8	7.7	8.2	7.6
320+30	6.9	6.7	7.2	7.4	7.3	7.2	7.7	7.1
323+00	7.7	7.2	8.8	8.2	8.0	8.0	8.4	8.4
326+00	8.5	7.7	10.0	9.0	9.2	9.2	9.1	9.1
329+00	9.3	8.2	11.1	9.6	10.2	10.2	9.8	10.1
331+60	9.3	7.9	11.5	9.6	10.6	10.2	9.8	10.1
335+06‡	10.0	8.2	12.6	10.3	11.0	10.8	10.4	10.7
338+48	10.7	8.5	13.5	10.9	11.6	11.9	11.0	11.2
342+00	11.4	9.1	14.5	11.7	12.1	12.4	11.7	11.9
345+00	11.5	10.2	14.5	11.7	12.2	↑	11.8	12.0
348+00	11.8	11.2	14.5	11.9	12.4	↑	12.1	12.1
352+00	12.8	12.8	14.4	12.8	12.8	↑	12.9	12.9
356+00	14.4	↑	14.4	↑	↑	↑	14.4	14.4
359+00	15.8	↑	↑	↑	↑	↑	↑	↑
365+00	17.8	↑	↑	↑	↑	↑	↑	↑
369+50	19.5	↑	↑	↑	↑	↑	↑	↑
372+39	23.7	↑	↑	↑	Same as base test	↑	↑	↑

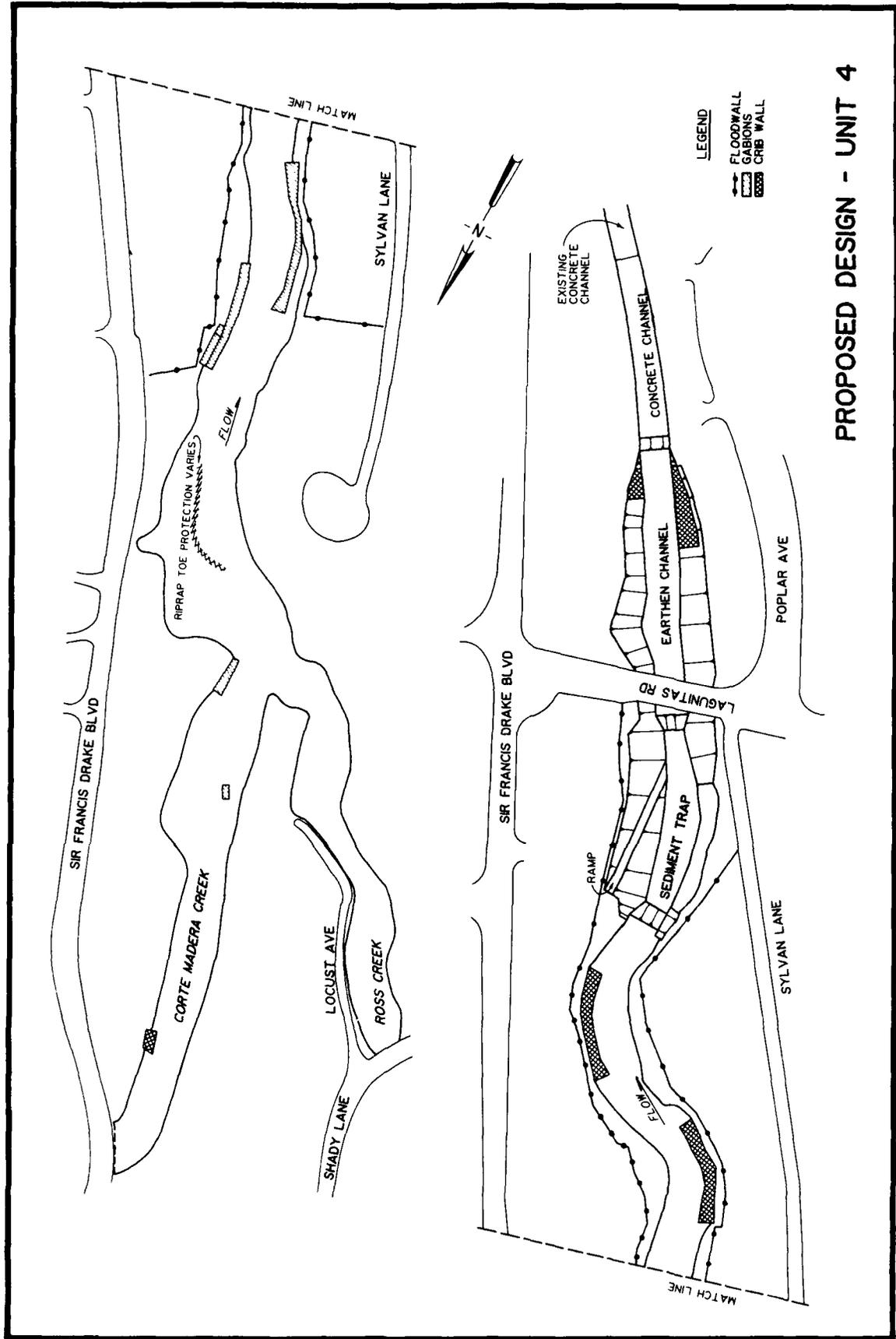
* Unit 4 design, 1982 flood season with 100-year-frequency flood in place of 4 January flood. Calculated $n = 0.022$ between stilling basin and sta 342+00. Downstream water-surface elevation equals mean sea level.

** Between stilling basin and sta 342+00.

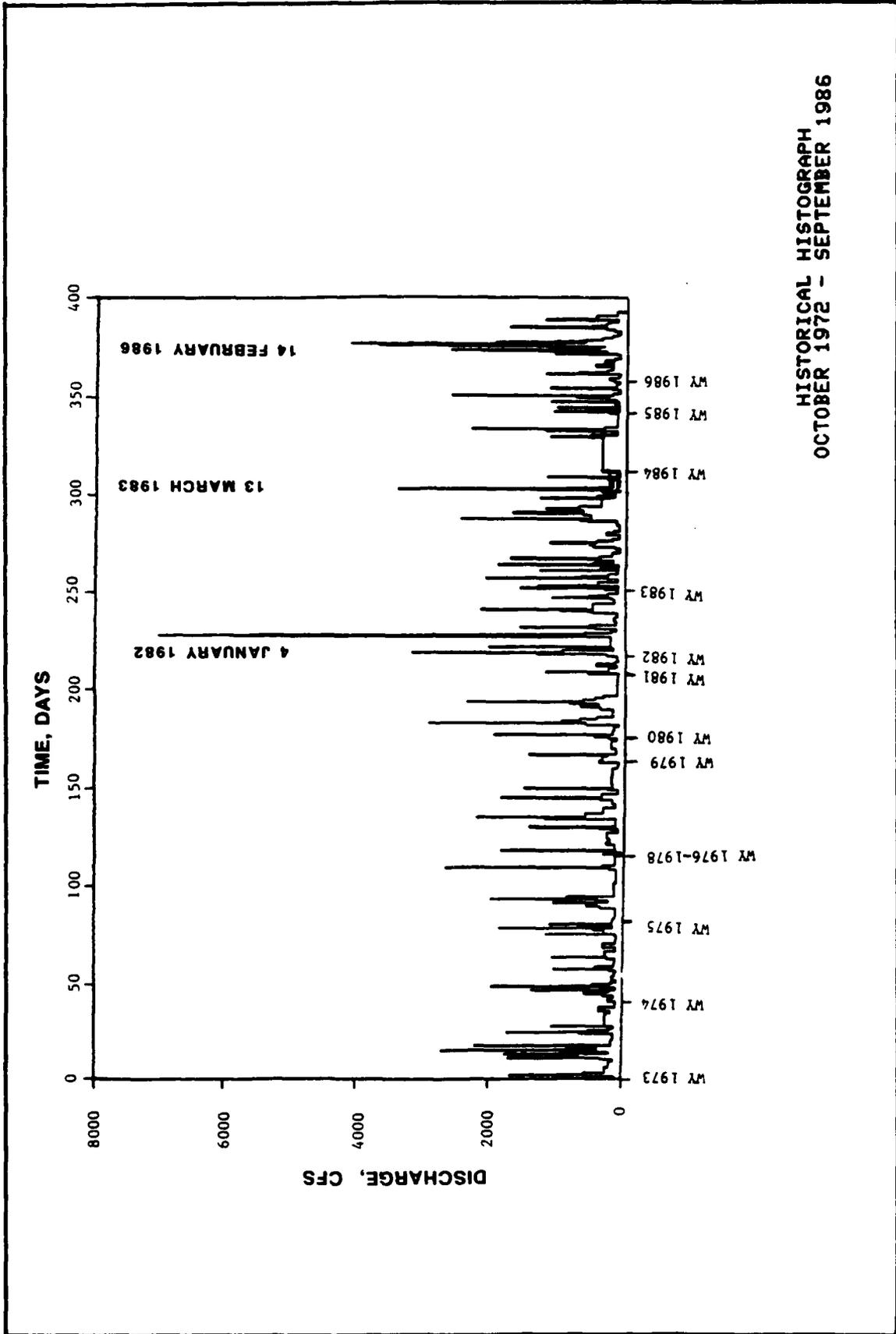
† Design flood placed at end of 1982 flood season.

†† Stilling basin at sta 319+05.

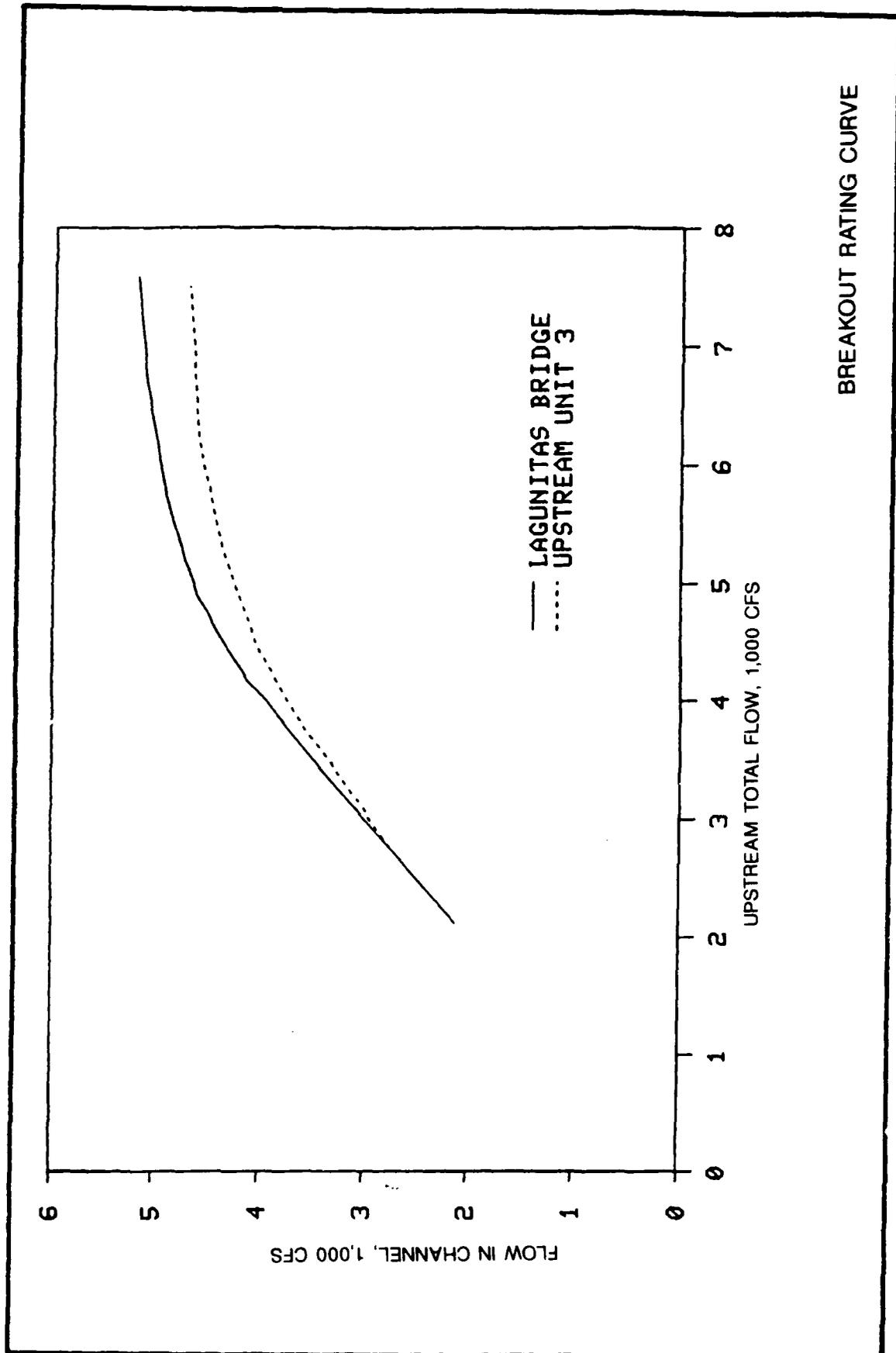
‡ College Avenue at sta 335+06.



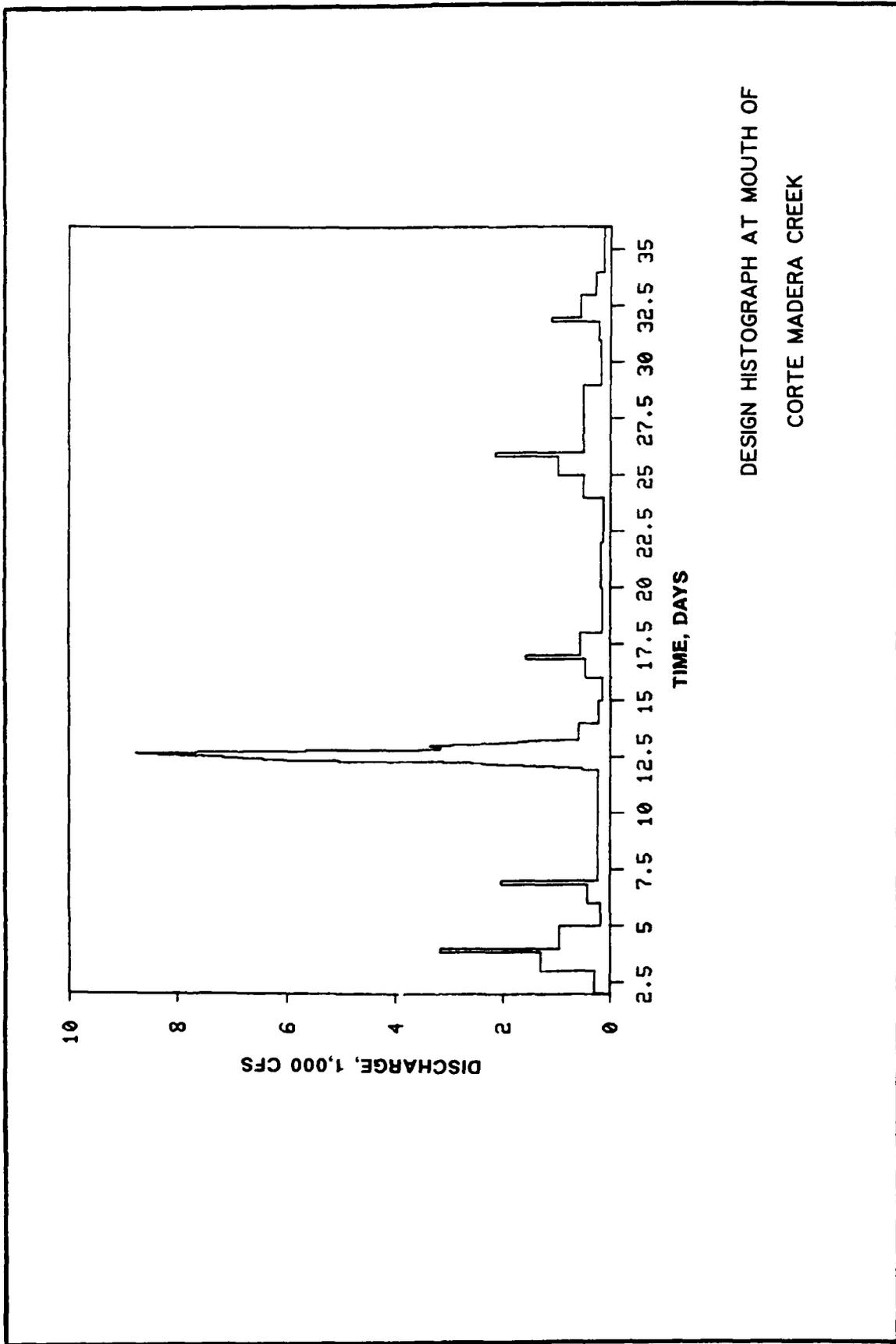
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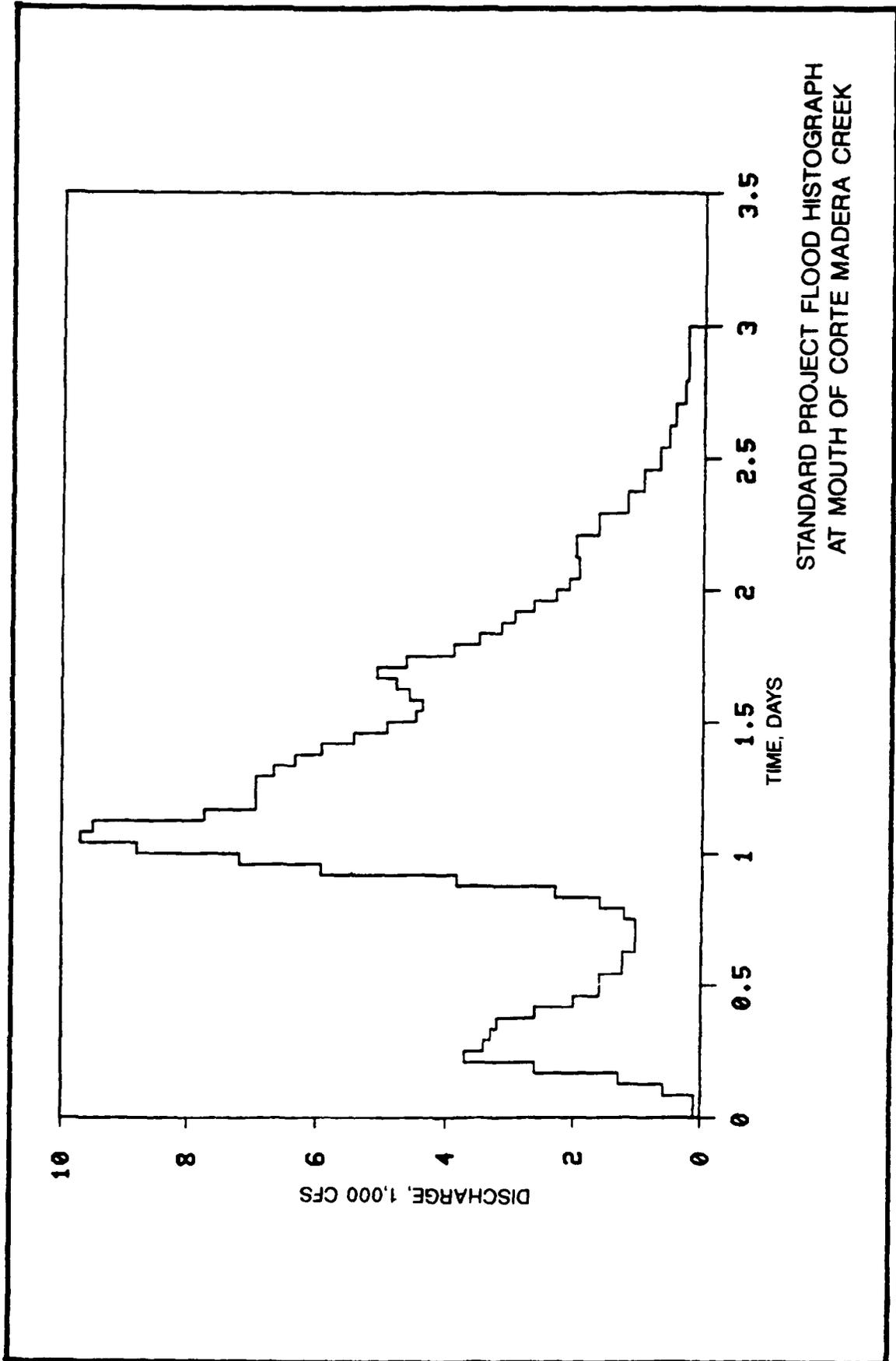
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OCTOBER 1972 - SEPTEMBER 1986



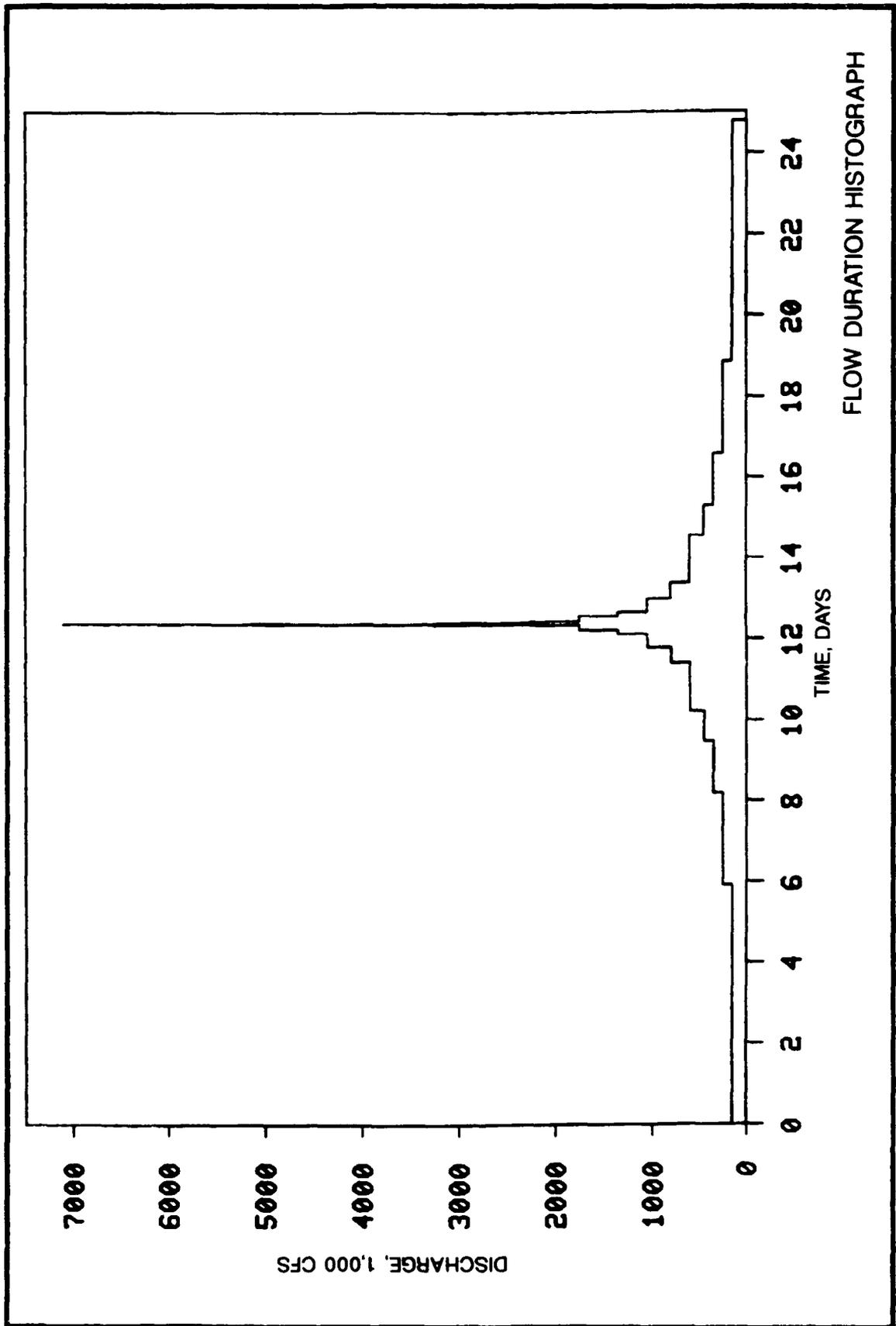
BREAKOUT RATING CURVE



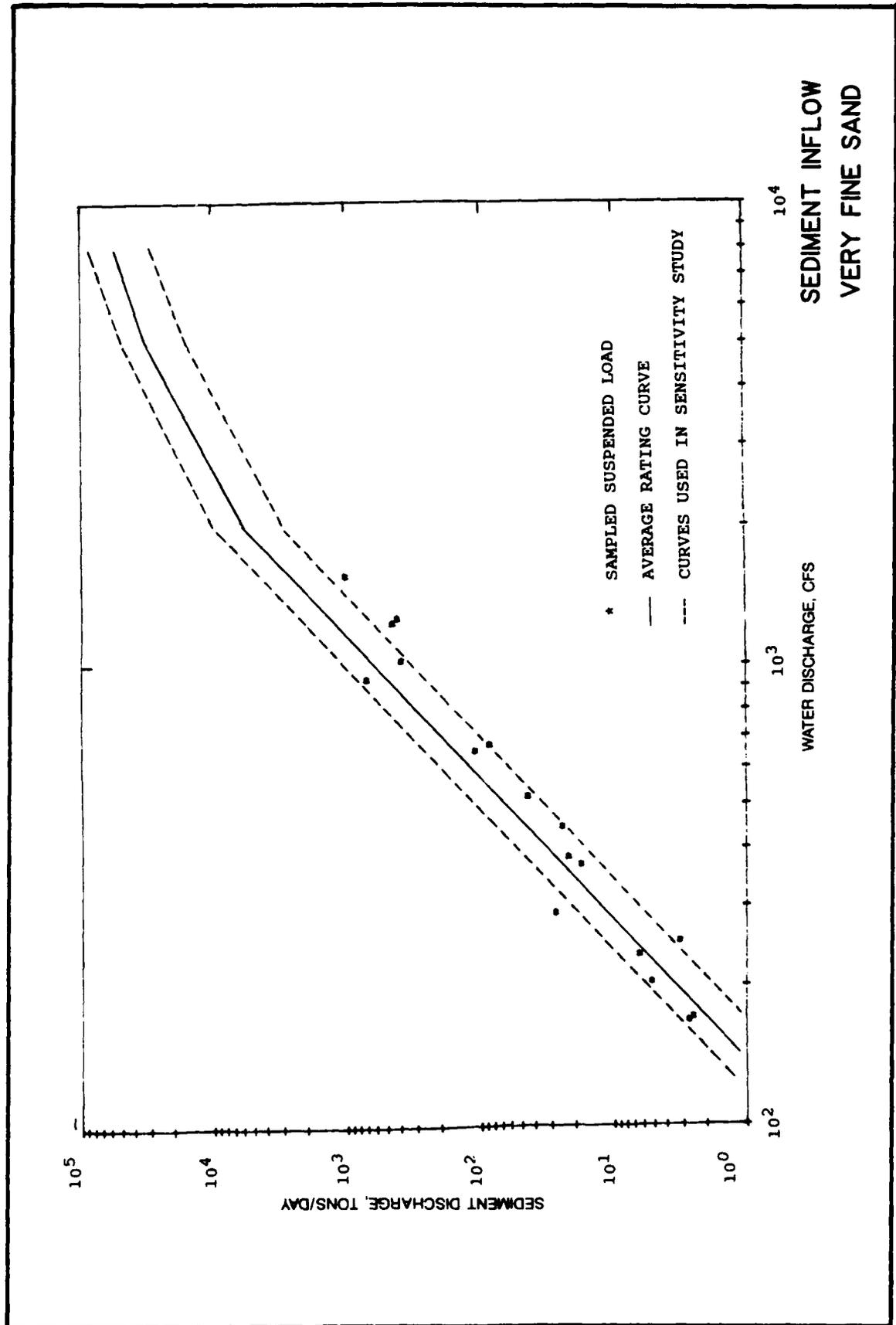
DESIGN HISTOGRAM AT MOUTH OF
CORTE MADERA CREEK

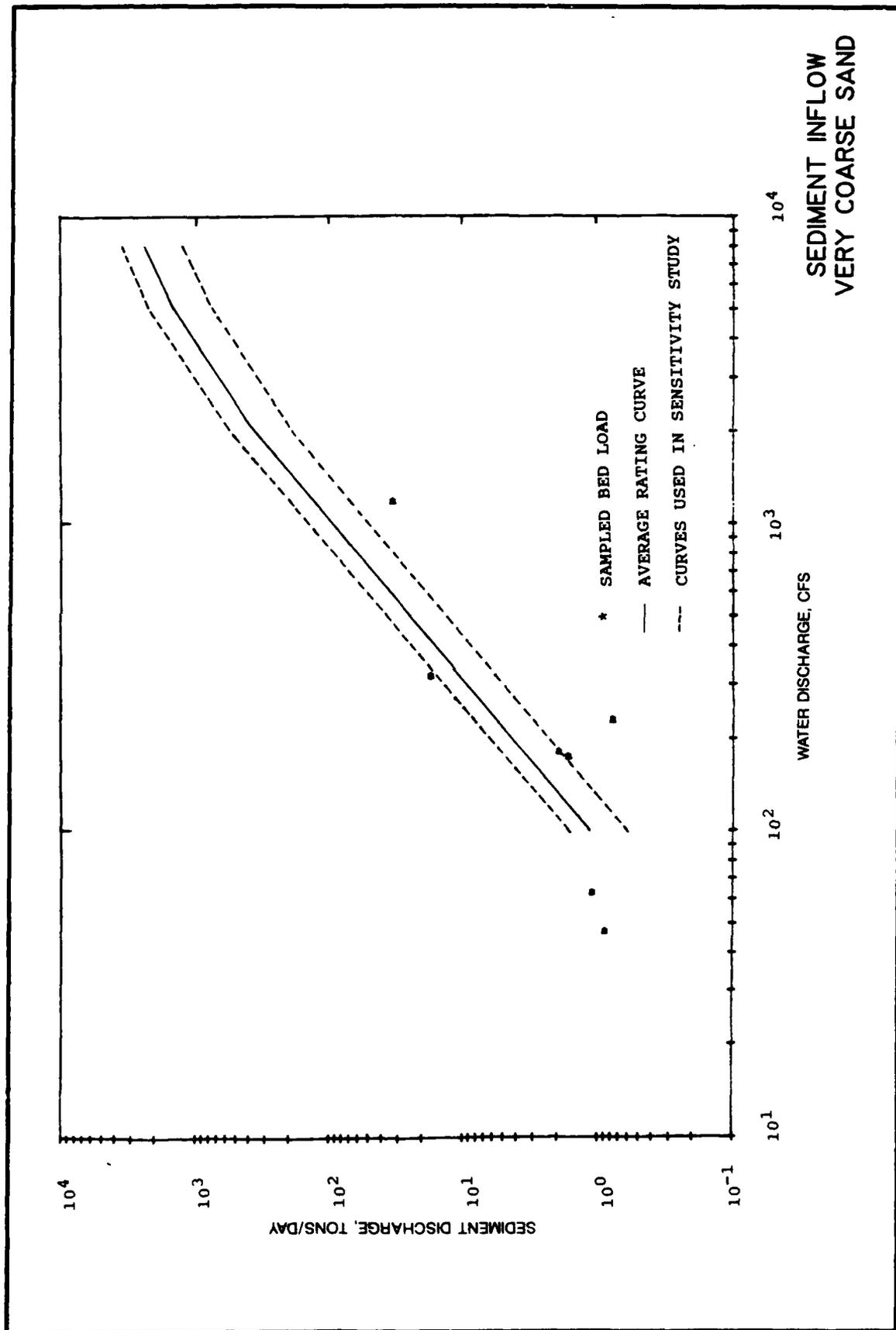


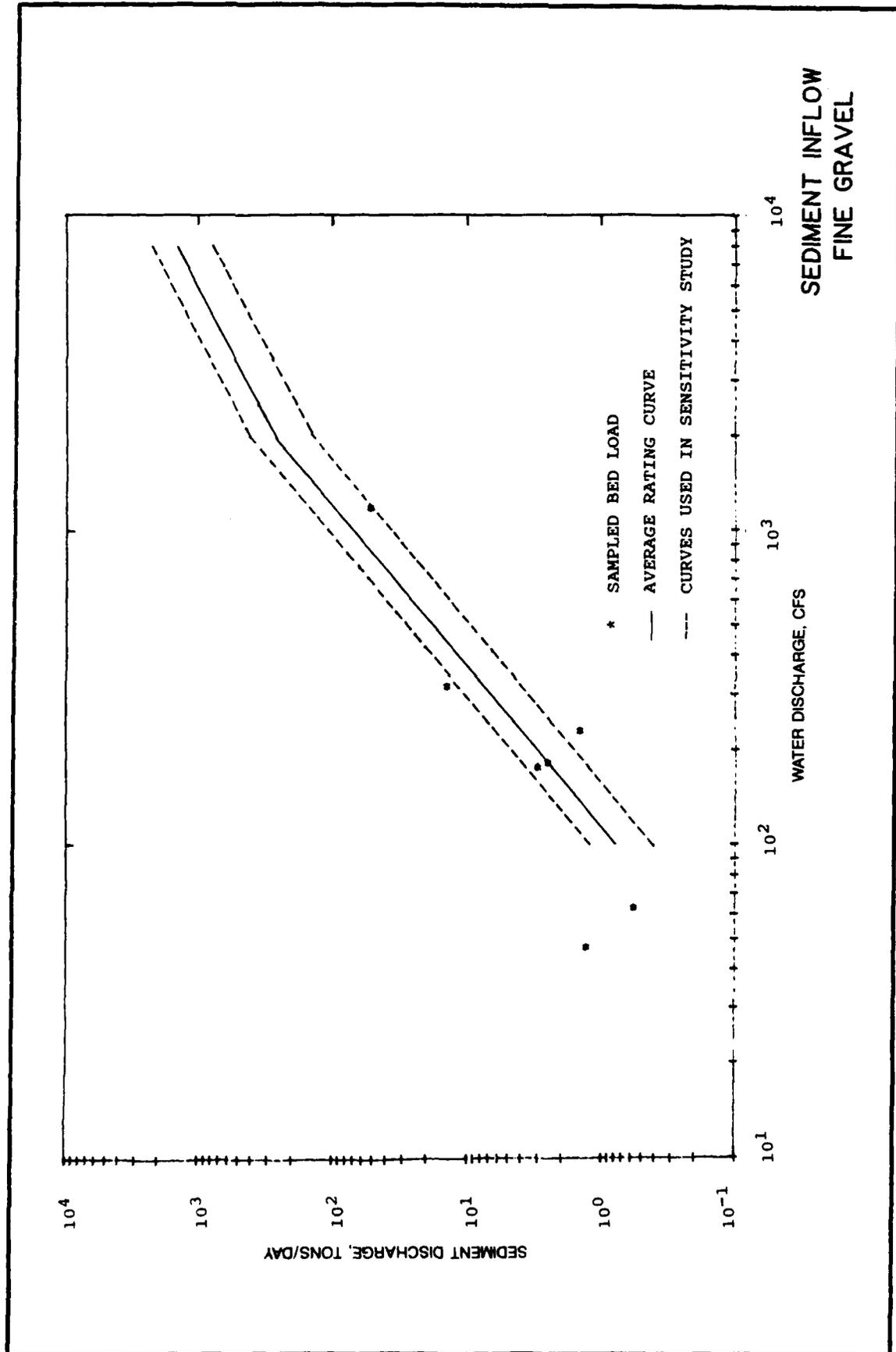
STANDARD PROJECT FLOOD HISTOGRAM
AT MOUTH OF CORTE MADERA CREEK

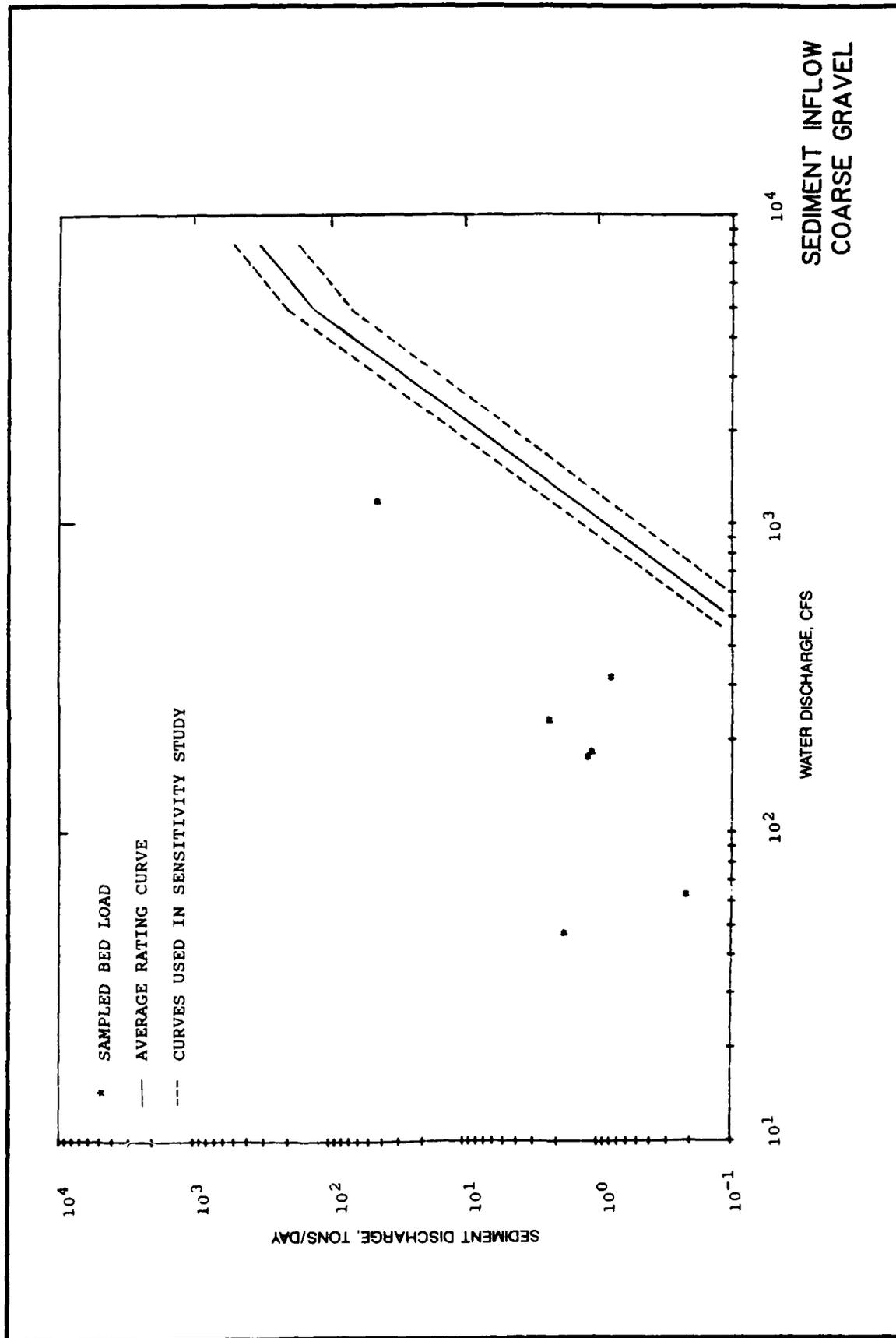


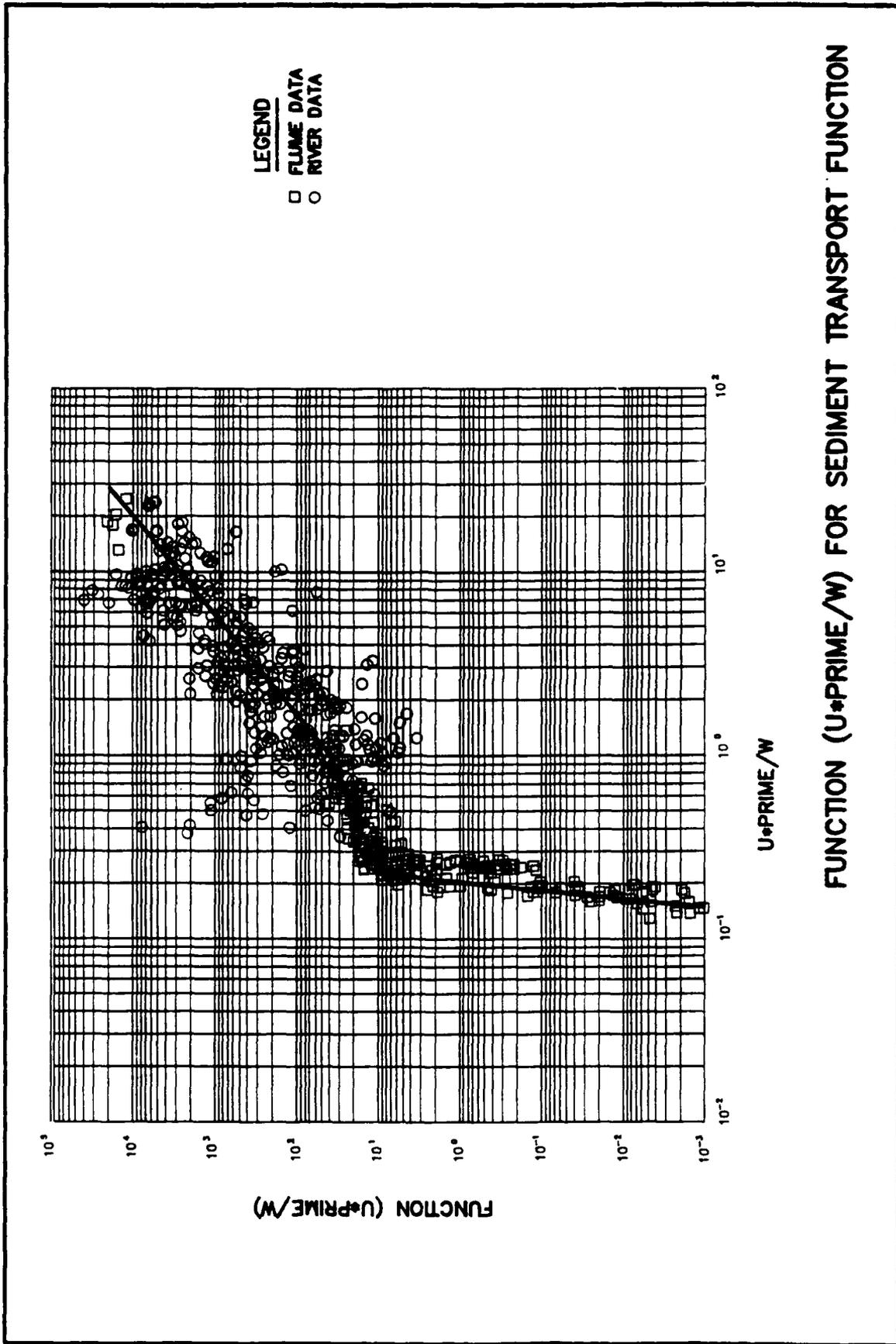
FLOW DURATION HISTOGRAM



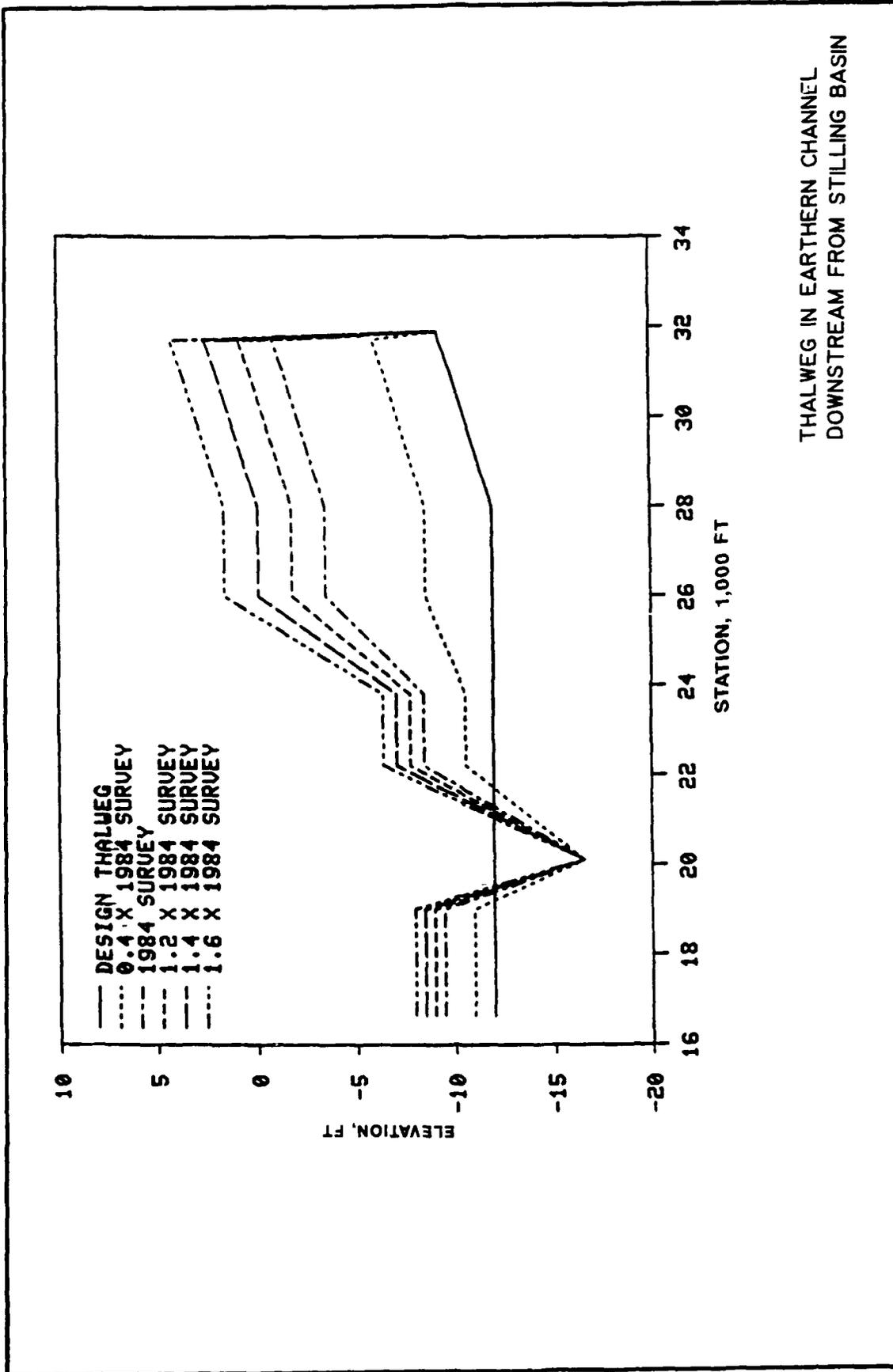




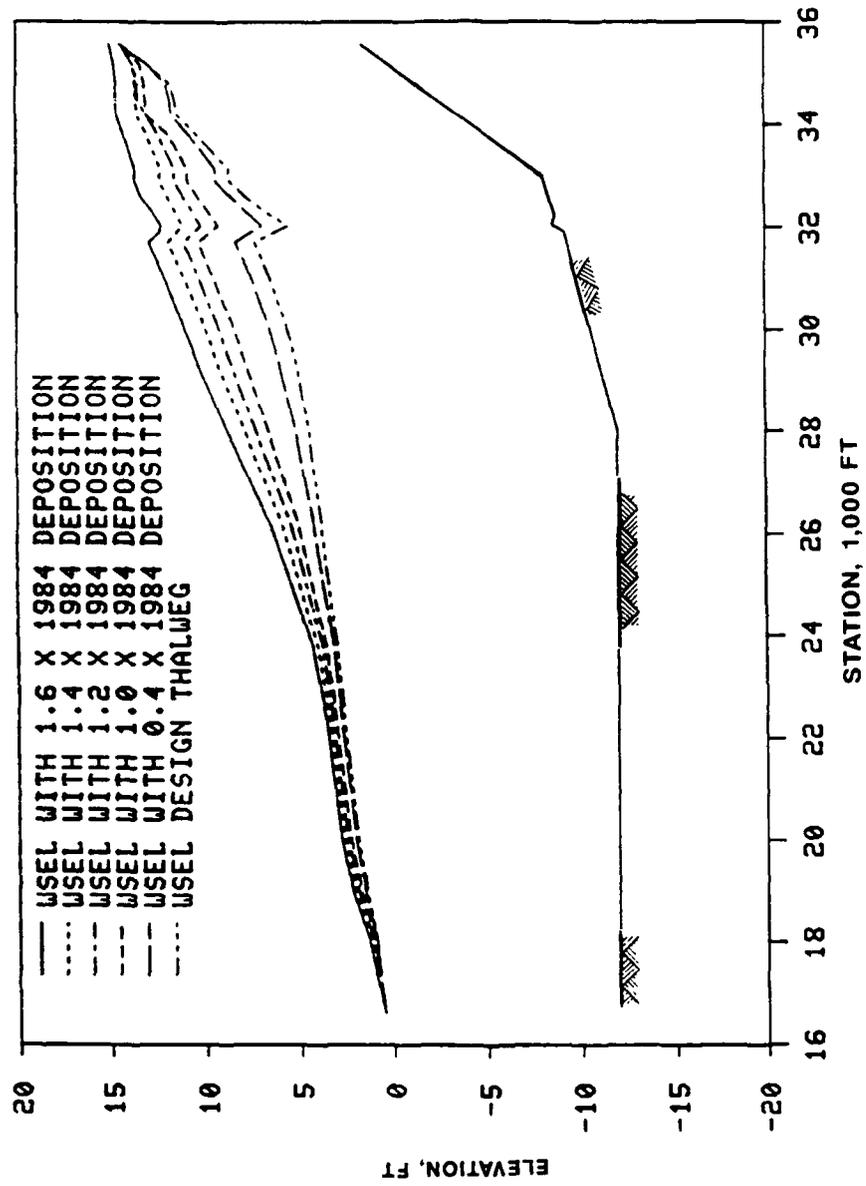




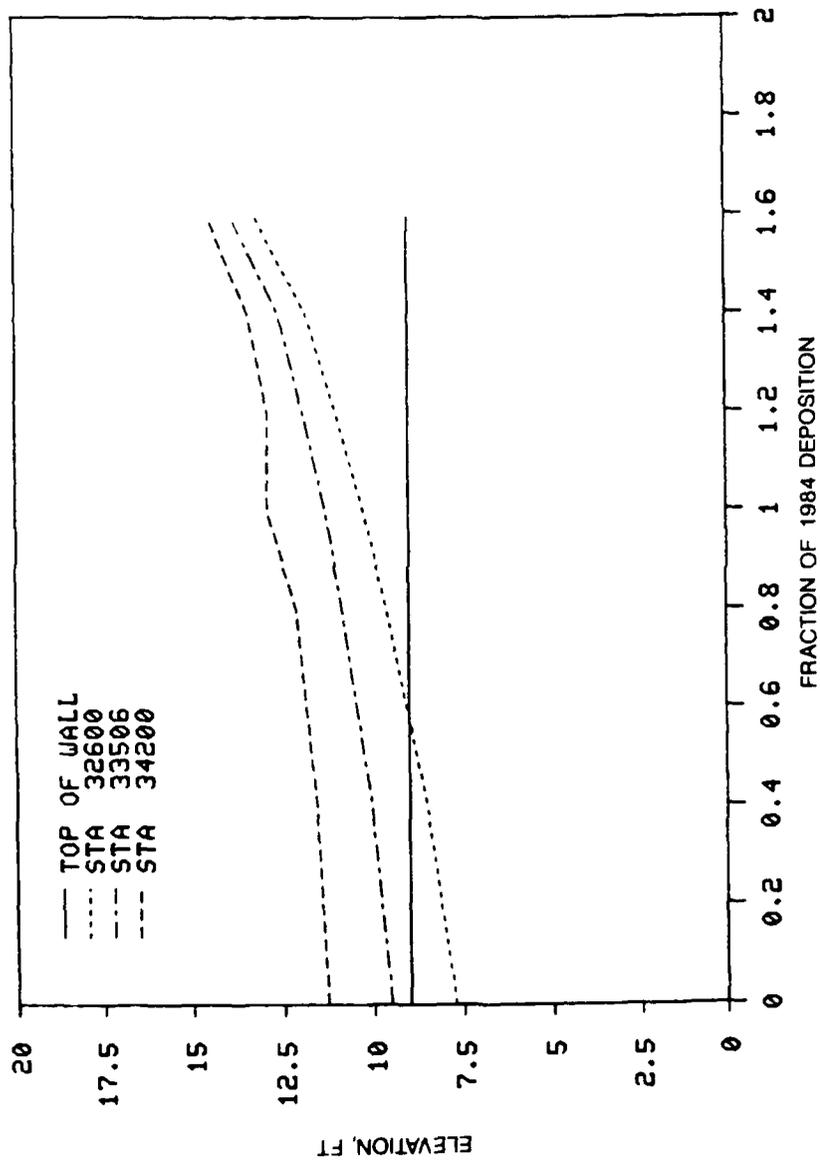
FUNCTION (U*PRIME/W) FOR SEDIMENT TRANSPORT FUNCTION



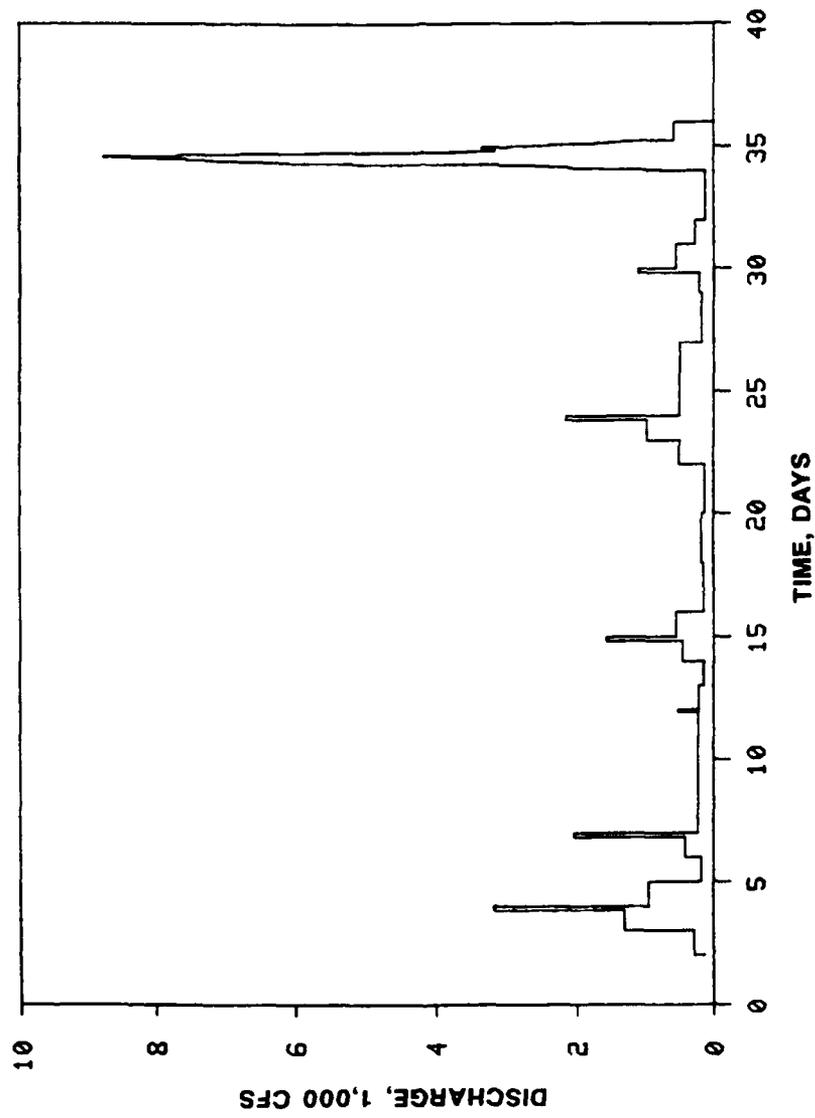
THALWEG IN EARTHEN CHANNEL
 DOWNSTREAM FROM STILLING BASIN



CALCULATED WATER-SURFACE ELEVATION
 PEAK OF 100-YEAR FREQUENCY FLOOD



EFFECT OF DOWNSTREAM DEPOSITION
ON 100-YEAR-FREQUENCY FLOOD
WATER-SURFACE ELEVATIONS IN CONCRETE CHANNEL



DESIGN HISTOGRAM WITH INCREASED ANTECEDENT FLOW
MOUTH OF CORTE MADERA CREEK

APPENDIX A: DESCRIPTION OF TABS-1 COMPUTER PROGRAM

1. The computer program TABS-1 calculates water-surface profiles and changes in the streambed profile. Water velocity, water depth, energy slope, sediment load, gradation of the sediment load, and gradation of the bed surface are also computed. Water-surface profile and sediment movement calculations are fully coupled using an explicit computation scheme. First, the conservation of energy equation is solved to determine the water-surface profile and pertinent hydraulic parameters (velocity, depth, width, and slope) at each cross section along the study reach:

$$\frac{\partial H}{\partial X} + \frac{\partial \left(\alpha \frac{V^2}{2g} \right)}{\partial X} = S \quad (A1)$$

where

H = water-surface elevation

X = direction of flow

α = coefficient for the horizontal distribution of velocity

V = average flow velocity

g = acceleration due to gravity

S = slope of energy line

In addition, the continuity of sediment material is expressed by

$$\frac{\partial G}{\partial X} + B \cdot \frac{\partial y_s}{\partial t} = q_s \quad (A2)$$

where

G = rate of sediment movement, cu ft/day

X = distance in direction of flow, ft

B = width of movable bed, ft

y_s = change in bed surface elevation, ft

t = time, days

q_s = lateral inflow of sediment, cu ft/ft/day

The third equation relates the rate of sediment movement to hydraulic parameters as follows:

$$G = f(V, y, B, S, T, d_{\text{eff}}, d_{\text{si}}, P_i) \quad (\text{A3})$$

where

y = effective depth of flow

T = water temperature

d_{eff} = effective grain size of sediment mixture

d_{si} = geometric mean of class interval

P_i = percentage of i^{th} size class in the bed

2. The numerical technique used to solve Equation A1 is commonly called the Standard Step Method. Equation A2 has both time and space domains. An explicit form of a six-point finite difference scheme is utilized. Several equations of the form of Equation A3 are available. These transport capacity equations are empirical and G is determined analytically.

3. Equation A2 is the only explicit equation, but it controls the entire analysis by imposing stability constraints. Several different computation schemes were tested, and the six-point scheme proved the most stable. No stability criteria have been developed for this scheme. The rule of thumb is to observe the amount of bed change during a single computation interval and reduce the computation time until that bed change is tolerable.

4. Oscillation in the bed elevation is a key factor in selecting a suitable computation interval. The computation time interval must be made short enough to eliminate oscillation. On the other hand, computer time increases as the computation interval decreases. The proper value to use is determined by successive approximations, running test cases, and observing the amount of bed change.

5. Several supporting equations are required in transforming the field data for the computer analysis. The Manning equation is used to evaluate friction loss. Average geometric properties are combined, using an average end area approach, into an average conveyance for the reach. Manning's roughness coefficients are entered for the channel and both overbanks and may be changed with distance along the channel, discharge, or stage. In the Corte Madera version of TABS-1, composite channel roughness was calculated from

calculated bed roughness and assigned sidewall roughness. Construction and expansion losses are calculated as "other" losses by multiplying a coefficient times the change in velocity head. All geometric properties are calculated from cross-section coordinates.

6. Only subcritical flow may be analyzed in the computer program; however, zones of critical or supercritical flow may occur within the study reach. The program treats supercritical zones as critical for determination of water-surface elevation, but calculates hydraulic parameters for sediment transport based on normal depth. Critical depth in a section with both channel and overbank is defined as the minimum specific energy for that section assuming a level water surface. Starting water-surface elevations can be input as a rating curve with stage and discharge, or stage can be set for each specific time interval. Steady-state conditions are assumed for each time interval, although the discharge may be changed to account for tributary inflow. A hydrograph is simulated by creating a histogram of steady-state discharges, using small time intervals when discharge variations are great and longer time intervals when changes in water and sediment discharges are small.

7. In some cases the temperature of water can be an important parameter in sediment transport and, consequently, may be prescribed with each water discharge in the hydrograph. Flexibility of input permits a value to be entered as needed to change from a previous entry.

8. Geometry is input into the numerical model as a series of cross sections similar to the widely used HEC-2 backwater program (US Army Engineer Hydrologic Engineering Center 1982*). A portion of the cross section is designated as movable and a dredging template may also be specified. Spacing of cross sections is somewhat more critical for HEC-6 than it is with HEC-2 because of numerical stability problems. Long reach lengths are desirable because reach length and computation interval are related. Very short time intervals may be required if excessive bed changes occur within a reach. No special provisions are available to calculate head losses at bridges. The contracted opening may be modeled such that scour and deposition are simulated during the passing of a flood event, but calculated results must be interpreted with the aid of a great deal of engineering judgment and sensitivity analysis.

* References cited in this appendix are included in the References at the end of the main text.

9. Four different sediment properties are required: (a) the total concentration of suspended loads and bedloads, (b) grain-size distribution for the total concentration, (c) grain-size distribution for sediment in the streambed, and (d) unit weight of deposits. A wide range of sediment material may be accommodated in the transport calculations (0.004 mm to 64 mm).

10. The usefulness of a calculation technique depends a great deal upon the coefficients that must be supplied. As in HEC-2, Manning's n values, contraction coefficients, and expansion coefficients must be provided to accomplish the water-surface profile calculations. Several other coefficients are required for sediment calculations as follows:

- a. The specific gravity and shape of sediment particles must be specified.
- b. The bed shear stress at which silt or clay particles begin to move is a required coefficient.
- c. The unit weight of silt, clay, and sand deposits is somewhat like a coefficient because of the difficulty in measuring. Also, the density changes with time.

11. All of the sediment-related coefficients have default values because sediment data seem to be much more scarce than hydraulic data. There are fewer sources for generalized coefficients. All of the default values should be replaced by field data where possible, and the input data are structured for such a process.

APPENDIX B: DETERMINATION OF DESIGN ROUGHNESS COEFFICIENTS

1. The numerical model calculated an average composite roughness coefficient of 0.030 in the concrete channel between the stilling basin and sta 342+00 for the peak of the January 1982 flood. The roughness elements contributing to this composite roughness coefficients were grain roughness on the bed, form roughness due to gravel movement, concrete wall roughness, tube-worm and barnacle deposits on the wall, and sinuosity. Analysis was made to determine the contribution of each element to the composite roughness coefficient. The results of the analysis were then used to estimate a design roughness coefficient with channel wall deposits removed.

2. The contributions of some of the roughness elements were determined using the same methods incorporated into the model's roughness algorithm. Grain roughness on the bed was calculated using the Limerinos equation:

$$n_{gr} = \frac{0.0926 (R)^{1/6}}{1.16 + 2.03 \log \left(\frac{R}{D_{84}} \right)} \quad (B1)$$

where

n_{gr} = Manning's roughness coefficient due to grain roughness

R = average hydraulic radius, ft

D_{84} = particle size of which 84 percent of the bed is finer, ft

Form roughness due to gravel movement was found to increase the bed roughness coefficient by 0.010 based on calculations using the Brownlie upper regime flow equation. A total roughness coefficient for the bed n_b was determined by adding the calculated grain and form roughness elements. Wall roughness was divided into two parts: the wall above mean sea level is smooth concrete and the wall below mean sea level has tubeworm and barnacle deposits. These deposits have an average thickness of about 1 in., which was assumed to be the effective roughness height. The Keulegan (1938)* velocity distribution equation for fully rough flow was used to calculate the roughness coefficient for the wall deposits:

* References cited in this appendix are included in the References at the end of the main text.

$$n_{lw} = \frac{1.486 R^{0.1667}}{32.6 \log (12.27 R/k_s)} \quad (B2)$$

where

n_{lw} = Manning's roughness coefficient for the lower wall

R = average hydraulic radius, ft

k_s = effective roughness height, ft

A Manning's roughness coefficient for the upper wall n_{uw} of 0.014 was assigned. A composite roughness coefficient was calculated from the following equation:

$$n = \left(\frac{P_b n_b^{1.5} + P_{lw} n_{lw}^{1.5} + P_{uw} n_{uw}^{1.5}}{P_b + P_{lw} + P_{uw}} \right)^{2/3} \quad (B3)$$

where

n = Manning's roughness coefficient

P = perimeter

b = subscript denoting the bed

lw = subscript denoting the lower wall

uw = subscript denoting the upper wall

The calculated composite roughness coefficient was increased by 0.004 to account for sinuosity. The total roughness coefficient calculated using this process agrees with the model's calculated roughness coefficient for the January 1982 flood. Calculated values are shown in Table B1.

3. With annual removal of sediment deposits in the concrete channel and the construction of Unit 4, the numerical model study determined that the channel would be free of sediment deposits during the peak of the design flood. An analysis by the US Army Engineer District, Sacramento, of the 1986 flood in the supercritical portion of the concrete channel determined that the composite roughness coefficient in the sediment-free concrete channel was 0.016. If the walls had a roughness coefficient of 0.014 (there are no wall deposits in this reach), then the bottom roughness coefficient would be 0.017. The higher roughness on the bed is attributed to abrasion and fish rests

indented into the channel invert. For the analysis of design roughness coefficients, a roughness coefficient of 0.017 was assigned to the sediment-free concrete channel bottom downstream from sta 342+00.

4. Annual removal of channel wall deposits was evaluated using the analytical values determined for the various roughness elements. The wall deposit removal was assumed to be 50 percent effective when the design flood occurred. Using Equation B3 to calculate the composite roughness coefficient and then increasing the result by 0.004 to account for sinuosity, a design roughness coefficient of 0.020 was calculated. For purposes of determining minimum freeboard, form roughness of 0.010 was added to the bottom roughness coefficient, and a total roughness coefficient of 0.026 was calculated. Values for each roughness element are shown in Table B1.

5. This analysis demonstrates that, with annual removal of sediment and wall deposits, design roughness coefficients in the lower portion of the concrete channel could be reduced. A reduction in design roughness coefficient from 0.022 to 0.020 was calculated. The coefficient recommended to determine minimum freeboard was reduced from 0.028 to 0.026. These recommended values were determined analytically using a systematic approach to determine the contribution of various roughness elements and should not be considered as experimentally verified values.

Table B1
Values Assigned to Roughness Elements with Removal of
Wall and Bed Deposits

	January 1982		Design		Minimum Freeboard	
	Wall	Bed	Wall	Bed	Wall	Bed
Sediment grain roughness	--	0.023	--	--	--	--
Form roughness	--	0.010	--	--	--	0.010
Concrete	0.014	--	0.0140	0.017	0.0140	0.017
Wall deposits	0.021	--	0.0175	--	0.0175	--
Composite		0.026		0.016		0.022
Sinuosity		<u>0.004</u>		<u>0.004</u>		<u>0.004</u>
Total		0.030		0.020		0.026